4 Determination of Hazards

Chapters 1 through 3 introduced retrofitting and guided the designer through the technical process of pre-selecting retrofitting techniques for consideration. In this chapter, the analyses necessary to determine the flood- and non-flood-related forces and other site-specific considerations that control the design of a retrofitting measure are presented. This information may be useful in determining which retrofitting alternatives are technically feasible, and in preparing BCAs for those alternatives. The analysis of hazards contributes to the design criteria for retrofitting measures, which are described in Chapter 5.

Retrofitting measures must be designed, constructed, connected, and anchored to resist flotation, collapse, and movement due to all combinations of loads and geotechnical conditions appropriate to the situation, including:

- flood-related hazards, such as hydrostatic and hydrodynamic forces, flood-borne debris impact forces, and site drainage considerations;
- site-specific flood-related hazards, such as alluvial fans, closed basin lakes, and movable bed streams;
- non-flood-related environmental loads, such as earthquake and wind forces; and
- site-specific soil or geotechnical considerations, such as soil pressure, bearing capacity, land subsidence, erosion, scour, and shrink-swell potential.

4.1 Analysis of Flood-Related Hazards

The success of any retrofitting measure depends on an accurate assessment of the flood-related forces acting upon a structure. Floodwater surrounding a building exerts a number of forces on the structure, including lateral and vertical hydrostatic forces, hydrodynamic forces, and debris impact forces. In addition, certain flood-related conditions may pose a danger and require evaluation (e.g., site drainage, lake flooding, erosion debris flows) (see Figure 4-1).

Standing water or slow moving water can induce horizontal hydrostatic forces (pressures) against a structure, especially when floodwater levels on different sides of the structure are not equal. Saturated soils beneath the ground surface also impose hydrostatic loads on foundation components.

Figure 4-1. Flood-related hazards (top left: alluvial fan; top right: moveable bed stream; bottom left: closed basin lake; bottom right: interior drainage)

Hydrodynamic forces result from the velocity flow of water against or around a structure. These velocity flows, if fast enough, are capable of destroying solid walls and dislodging buildings with inadequate foundations. Impact loads are imposed on the structure by water-borne objects and their effects become greater as the velocity of flow and the weight of the objects increase. The basic equations for analyzing and considering these flood-related forces are provided in this chapter.

NOTE

Additional information concerning the determination of flood-related forces is available in Section 5 of ASCE 7, *Minimum Design Loads for Buildings and Other Structures* (2010).

Minimum standards for flood-resistant design are available in ASCE 24, *Flood Resistant Design and Construction Standard* (2005).

Minimum standards for flood-resistant design may be found in *Minimum Design Loads for Buildings and Other Structures* (ASCE 7) and *Flood Resistant Design and Construction* (ASCE 24). Equations for calculating the aforementioned forces for flood-related hazards can be found in technical publications from FEMA, such as FEMA P-55, *Coastal Construction Manual* (FEMA, 2011). FEMA P-55 provides guidance for designing and constructing residential buildings in coastal areas that will be more resistant to the damaging effects of natural hazards. The focus of this manual, FEMA 259, is on new residential construction and substantial improvement to existing residential buildings, principally detached single-family homes, attached single-family homes (townhouses), and low-rise (three-story or less) multi-family buildings.

4.1.1 Determining Flood Elevations

Determining the expected flood depth at a site is critical for the overall determination of flood-related hazards. The method for making this determination can vary depending on whether the site is subject to riverine or coastal flooding.

4.1.1.1 Riverine Areas

One method of determining the 100-year water-surface elevation involves using a DFIRM panel or a FIRM panel. The DFIRM or FIRM panel identifies the specific flood zone(s) and BFEs of the project area in question. For simplicity purposes, this manual, FEMA 259, determines flood depths using the DFIRM. On most DFIRMs, floodplain limits are delineated for the 1- and 0.2-percent-annual-chance flood. As an example, Figure 4-2 shows the portion of a community's DFIRM where a subject house is located.

NOTE

Note that for maps with small scales (greater than 1"=400'), converting feet to inches can introduce inaccuracies in locating the home and in specifying the flood elevations impacting the site.

In this example, the location of the house was determined by measuring the distance from the intersection of Anderson Drive and Shaftsberry Court. The house is located approximately 325 feet southeast of the intersection. Converting this distance to the map's scale (1 inch equals 500 feet), the house is 0.65 inch along Shaftsberry Court from its intersection with Anderson Drive.

The blue-dotted shading on the map represents the 100-year floodplain. The black-dotted shading denotes the 500-year floodplain. The house is located within the 100-year floodplain, in between the two wavy lines labeled 214. These lines denote

Figure 4-2. House and stream location on the DFIRM

the 100-year flood elevation at that location of Big Branch (Stream 21). Therefore, the 100-year flood elevation affecting the house in this example is 214 feet, based on the NAVD.

Flood elevations for the other frequencies are shown on the stream's water-surface profile in the FIS. For the above example, the position of the house on Big Branch (Stream 21) was determined by using the cross section line perpendicular to the stream labeled 023 as a reference point and measuring approximately 25 feet or 0.05 inch south on the DFIRM. The location of the stream is shown in Figure 4-2.

The house can be located on the Big Branch (Stream 21) flood profile (Figure 4-3) and measured 0.125 inch downstream of cross section 023 (25 divided by 200 feet per inch, which is the horizontal scale of the profile). This location is marked as the point on Big Branch (Stream 21) with water-surface elevations equivalent to the house. The elevations on the profile at this point are 207.0, 213.9, and 219.0 feet for the 10-, 100-, and 500-year floods, respectively. The bottom of the Big Branch (Stream 21) channel shown on the profile is at 191.7 feet.

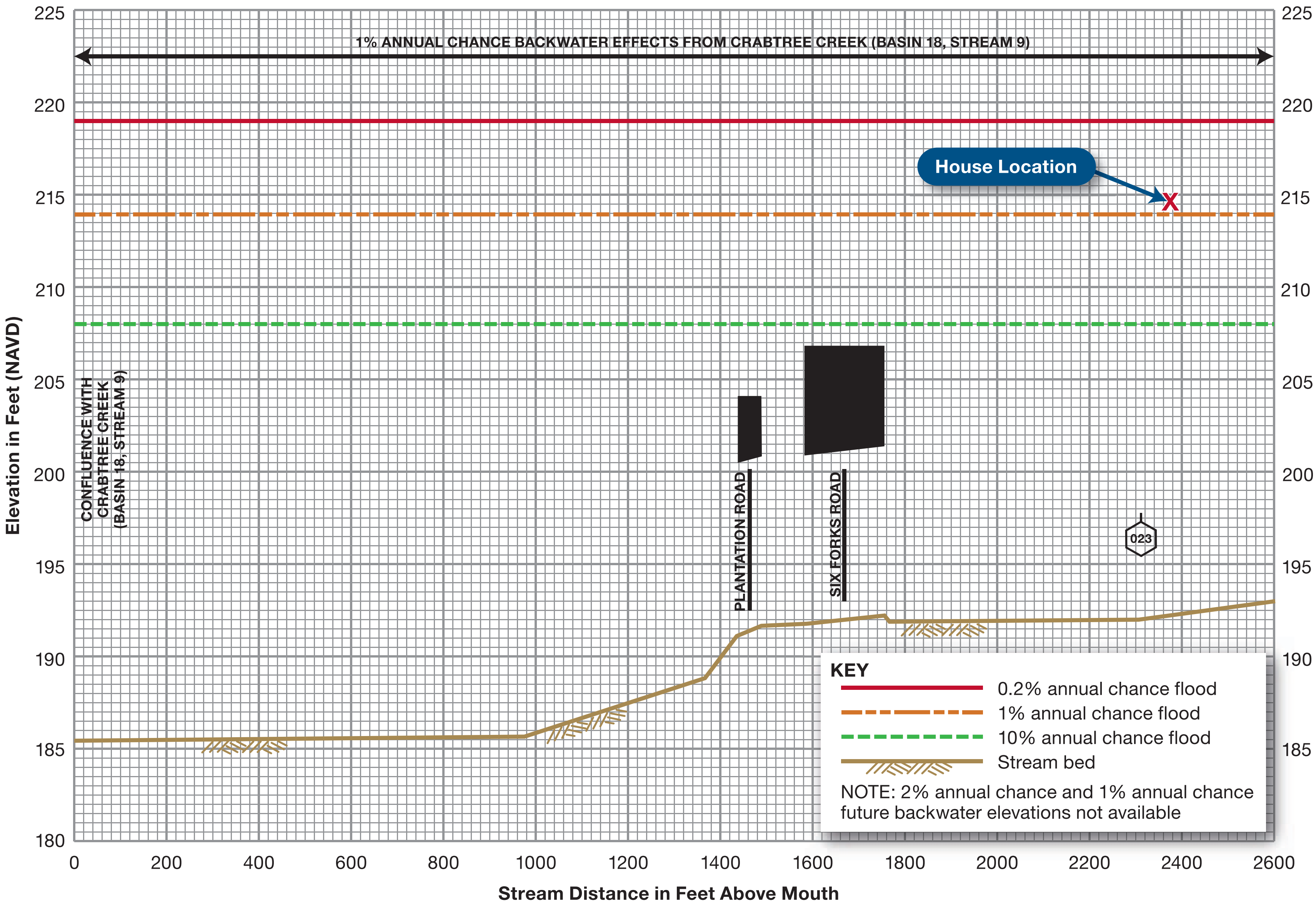


Figure 4-3. House location on flood profile for Big Branch (Stream 21)

Since Big Branch (Basin 18, Stream 21) is mapped as a Zone AE and has a floodway, a floodway data summary table can be obtained from the FIS. Table 4-1 depicts the floodway data table for this example. The regulatory BFE is listed as 213.9 feet below.

Table 4-1. Floodway Data Summary Table for Big Branch (Stream 21)

Flooding Source		Floodway			Base Flood Water-Surface Elevation (ft NAVD 88)			
Cross Section	Distance[a]	Width (ft)	Section Area (ft^2)	Mean Velocity (ft/sec)	Regulatory	Without Floodway	With Floodway	Increase
Big Branch (Basin 10, Stream 8)								
013	1,255	110	419	3.3	254.0	253.6[b]	254.6	1.0
054	5,360	70	156	6.5	276.3	276.3	276.3	0.0
Big Branch (Basin 18, Stream 21)								
023	2,308	140	1,193	3.0	213.9	209.0[c]	209.4	0.4
028	2,765	110	1,024	3.5	213.9	209.5[c]	209.8	0.3
034	3,358	120	773	4.6	213.9	210.1[c]	210.6	0.5
043	4,297	70	439	7.6	213.9	214.0[c]	214.9	0.9
048	4,813	40	430	7.8	220.1	220.1	220.2	0.1
058	5,774	100	1,918	2.1	232.8	232.8	233.5	0.7

SOURCE: FEMA FIS REPORT FOR WAKE COUNTY, NC

a. Feet above mouth

b. Elevation computed without consideration of backwater effects from Little River (Basin 10, Stream 1)

c. Elevation computed without consideration of backwater effects from Crabtree Creek (Basin 18, Stream 9)

4.1.1.2 Coastal Areas

In coastal areas, the determination of the expected water surface elevation for the various RI floods is made by locating the structure and its flooding source on the DFIRM, identifying the corresponding flooding source/location row on the summary of stillwater elevation table, and selecting the appropriate elevation for the RI in question.

As an example, consider a building located on Marsh Bay Drive (as depicted on Figure 4-4). From the DFIRM, we can identify the flooding source as the Atlantic Ocean. The marked structure is located in a Zone AE, and has a BFE of 14 feet. In coastal areas, the BFE is equal to the stillwater elevation plus the associated wave height.

A review of the entire area map for the FIS would indicate the structure on Marsh Bay Drive is located between transect lines 46 and 47.

CROSS REFERENCE

A detailed discussion of the methodologies involved in computing wave heights and runup is beyond the scope of this manual. For more information, refer to:

- *Guidance for Wave Elevation Determination and V Zone Mapping* (FEMA, 2003)
- *Guidance for Pacific Coast of the United States* (FEMA, 2005a)
- *Guidance for Atlantic and Gulf Coasts of the United States* (FEMA, 2007)
- FEMA P-55, *Coastal Construction Manual,* Fourth Edition, (FEMA, 2011)

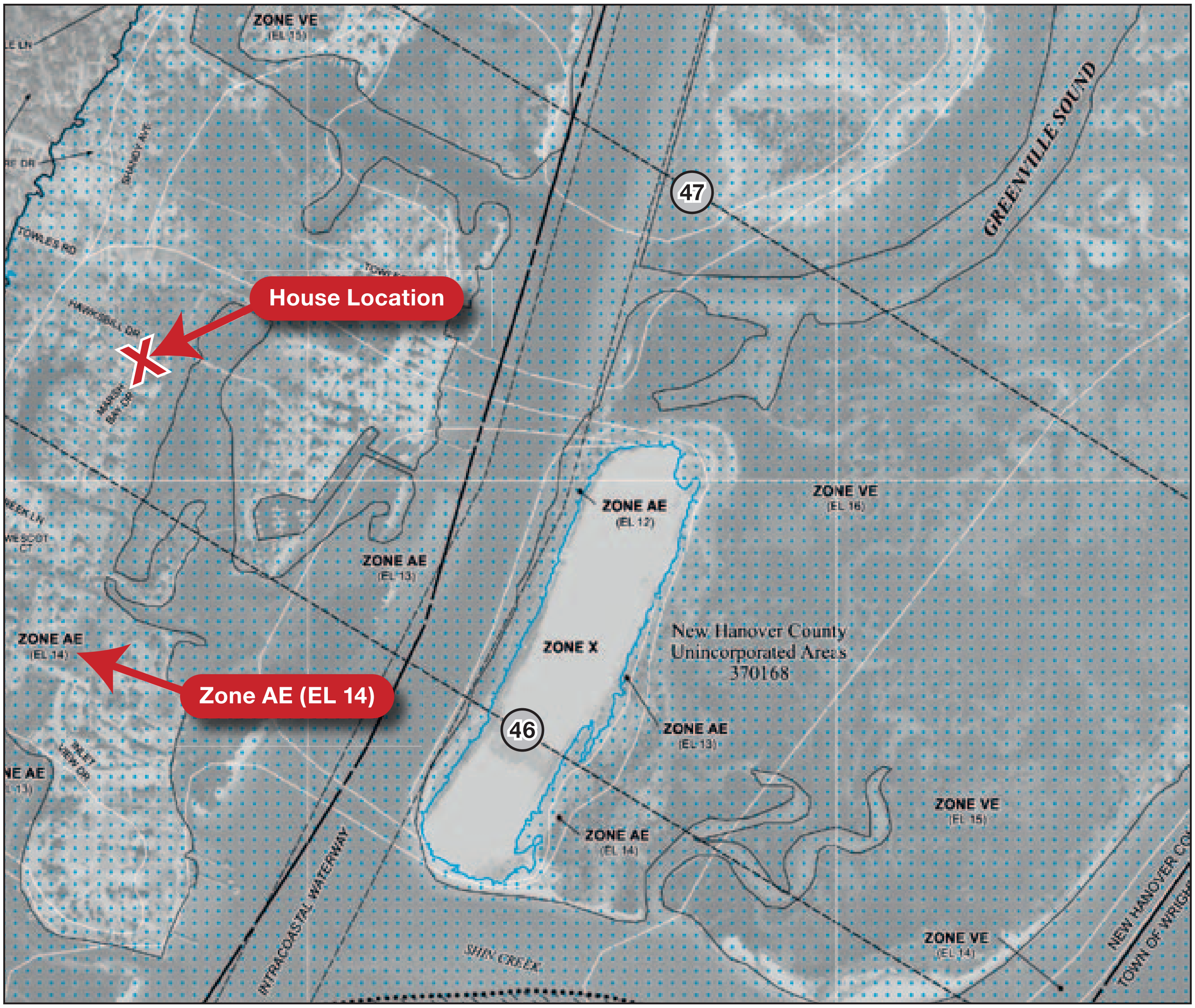


Figure 4-4. Coastal DFIRM showing house location and flood elevation

This flooding source/location is on the summary of stillwater elevations table (Table 4-2). From this table, the identified transect numbers are used to determine the stillwater flood elevations. Stillwater flood elevations of 5.7, 8.7, 12.2, and 12.4 feet in NAVD are identified for the 10-, 50-, 100-, and 500-year frequency floods (10-percent, 2-percent, 1-percent, and 0.2-percent-annual-exceedance probabilities), respectively.

Table 4-2. Summary of Coastal Analysis for the Atlantic Ocean Flooding Source

Transect		Stillwater Elevation in ft NAVD 88				Wave Runup Analysis Zone Designation and BFE in ft NAVD 88	Wave Height Analysis Zone Designation and BFE in ft NAVD 88	Primary Frontal Dune Identified
No.	Location	10% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance			
45	Approximately 1.87 miles southeast of the intersection of Orchard Trc and Masonboro Sound Rd	5.7	8.7	12.2	12.4	N/A	VE 14-19 AE 12-14	Yes
46	Approximately 690 ft southeast of intersection of Jack Parker Blvd and S Lumina Ave	5.7	8.7	12.2	12.4	N/A	VE 14-19 AE 12-14	Yes
47	Approximately 580 ft southeast of the intersection of S Lumina Ave and Sunset Ave	5.7	8.7	12.2	12.4	N/A	VE 14-19 AE 12-14	Yes
48	Approximately 550 ft east of the intersection of S Lumina Ave and Bridgers St	5.7	8.7	12.2	12.4	N/A	VE 14-19 AE 12-14	Yes

SOURCE: FEMA FIS REPORT FOR NEW HANOVER COUNTY, NC

4.1.2 Flood Forces and Loads

Floodwater can exert a variety of forces on a building. This section describes these forces, which include hydrostatic, saturated soil, hydrodynamic, debris impact, and erosive forces and illustrates how they are computed.

4.1.2.1 Flood Depth and Floodproofing Design Depth

After gathering flood data from the riverine or coastal DFIRM and FIS, it is possible to compute the depth of flooding at a structure for any of the RIs defined along the flooding source. Flood depth can be computed by subtracting the lowest ground surface elevation (grade) adjacent to the structure from the flood elevation for each flood frequency, as shown in Equation 4-1. Sample calculations using these equations are presented in Appendix C.

Many communities have chosen to exceed minimum NFIP building elevation requirements, usually by requiring freeboard above the BFE, but sometimes by regulating to a more severe flood than the base flood. In this manual, "design flood elevation" refers to the locally adopted regulatory flood elevation. If a community regulates to minimum NFIP requirements, the DFE is identical to the BFE. If a community has chosen to exceed minimum NFIP elevation requirements, the DFE exceeds the BFE. The DFE is always

EQUATION 4-1: FLOOD DEPTH

$$d = FE - GS \qquad \text{(Eq. 4-1)}$$

where:

d = depth of flooding (ft)

FE = flood elevation for a specific flood frequency (ft)

GS = lowest ground surface elevation (grade) adjacent to a structure (ft)

NOTE

When computing flood depth, be sure to use the lowest ground surface adjacent to the structure in question as shown in Figure 4-5.

equal to or greater than the BFE and includes wave effects. One common way of specifying the DFE, using freeboard above BFE, is illustrated in Equation 4-2. Communities incorporate freeboard with the intent that structures be elevated above this level, but they may or may not intend that all design loads be based on this elevation (many communities require freeboard to achieve flood insurance premium savings or Community Rating System [CRS] discount points). The rationale for freeboard adoption should be investigated before flood loads are calculated.

EQUATION 4-2: COMMON DEFINITION OF DESIGN FLOOD ELEVATION

$$DFE = FE + f \qquad \text{(Eq. 4-2)}$$

where:

DFE = design flood elevation (ft)

FE = flood elevation for a specific flood frequency (ft)

f = factor of safety (freeboard), typically a minimum of 1.0 ft

Determining the floodproofing design depth at the structure is very important for the flood load calculation process. Nearly every other flood load parameter or calculation (e.g., hydrostatic load, hydrodynamic load, vertical hydrostatic load, debris impact load, and local scour depth) depends directly or indirectly on the floodproofing design depth. The floodproofing design depth *(H)* is the difference between the DFE and the lowest grade adjacent to the structure (Figure 4-5). This computation is shown in Equation 4-3.

EQUATION 4-3: FLOODPROOFING DESIGN DEPTH

$$H = DFE - GS \qquad \text{(Eq. 4-3)}$$

where:

H = floodproofing design depth over which flood forces are considered (ft)

DFE = design flood elevation (ft)

GS = lowest ground surface elevation (grade) adjacent to the structure (or other reference feature such as a slab or footing) (ft)

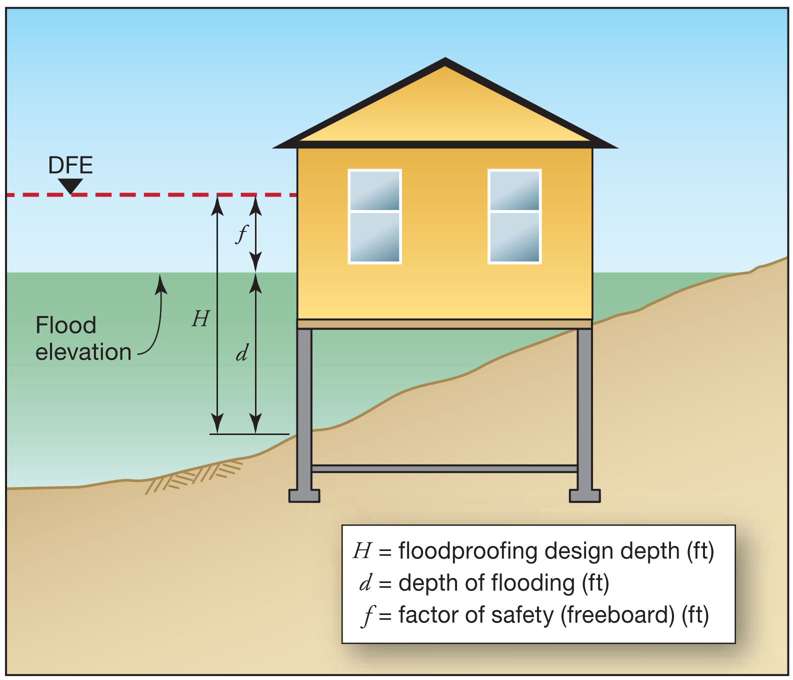


Figure 4-5. Flood depth and design depth

4.1.2.2 Hydrostatic Forces

The pressure exerted by still and slow moving water is called "hydrostatic pressure." During any point of floodwater contact with a structure, hydrostatic pressures are equal in all directions and always act perpendicular to the surface on which they are applied. Pressures increase linearly with depth or "head" of water above the point under consideration. The summation of pressures over the surface under consideration represents the load acting on that surface. For structural analysis, hydrostatic forces, as shown in Figures 4-6 and 4-7, are defined to act:

- vertically downward on structural elements such as flat roofs and similar overhead members having a depth of water above them;

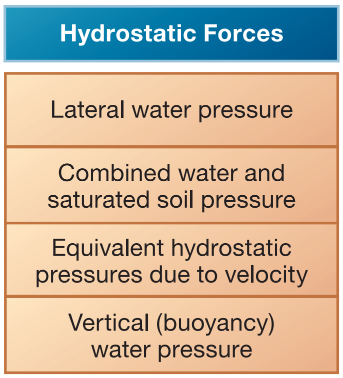


Figure 4-6. Hydrostatic forces

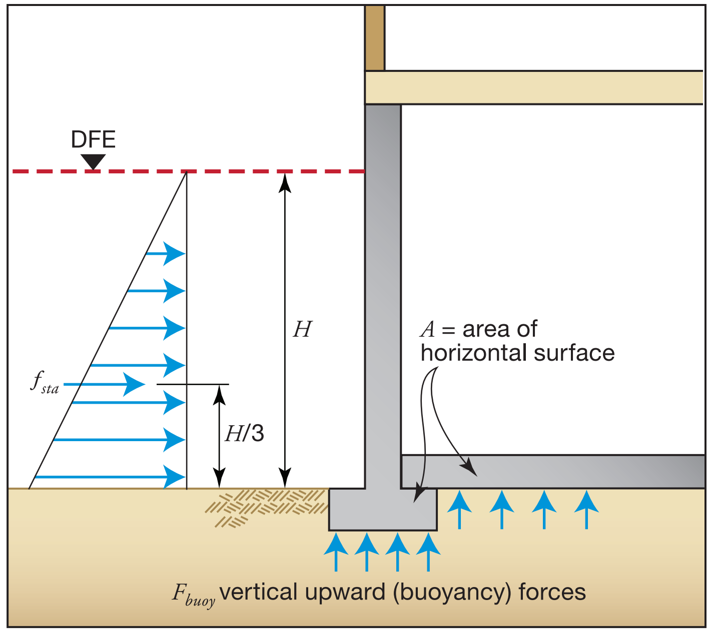


Figure 4-7. Diagram of hydrostatic forces

- vertically upward (uplift) from the underside of generally horizontal members such as slabs, floor diaphragms, and footings (also known as buoyancy); and
- laterally, in a horizontal direction on walls, piers, and similar vertical surfaces. (For design purposes, this lateral pressure is generally assumed to act on the receiving structure at a point one-third of the water depth above the base of the structure or two-thirds of the altitude from the water surface, which correlates to the center of gravity for a triangular pressure distribution.)

Hydrostatic forces include lateral water pressures, combined water and soil pressures, equivalent hydrostatic pressures due to velocity flows, and vertical (buoyancy) water pressures. The computation of each of these pressures is illustrated in the sections that follow.

For the purpose of this document, it has been assumed that hydrostatic conditions prevail for stillwater and water moving with a velocity of less than 10 ft/sec.

Hydrostatic loads generated by velocities up to 10 ft/sec may be converted to an equivalent hydrostatic load using the conversion equation, Equation 4-8, presented later in this chapter.

4.1.2.3 Lateral Hydrostatic Forces

The basic equation for analyzing the lateral force due to hydrostatic pressure from standing water above the surface of the ground is illustrated in Equation 4-4.

EQUATION 4-4: LATERAL HYDROSTATIC FORCES

$$f_{sta} = \frac{1}{2} P_h H = \frac{1}{2} \gamma_w H^2 \qquad \text{(Eq. 4-4)}$$

where:

f_{sta} = hydrostatic force from standing water (lb/lf) acting at a distance $H/3$ above ground

P_h = hydrostatic pressure due to standing water at a depth of H (lb/ft^2), ($P_h = \gamma_w H$)

γ_w = specific weight of water (62.4 lb/ft^3 for fresh water and 64.0 lb/ft^3 for saltwater)

H = floodproofing design depth (ft)

4.1.2.4 Saturated Soil Forces

If any portion of the structure is below grade, saturated soil forces must be included in the computation in addition to the hydrostatic force. The equivalent fluid pressures for various soil types are presented in Tables 4-3 and 4-4. The equivalent fluid weight of saturated soil is not the same as the effective weight of saturated soil. Rather, the equivalent fluid weight of saturated soil is a combination of the unit weight of water and the effective saturated weight of soil.

Table 4-3. Effective Equivalent Fluid Weight of Submerged Soil and Water

Soil Type*	Equivalent Fluid Weight of Submerged Soil and Water (lb/ft^3)
Clean sand and gravel (GW, GP, SW, SP)	75
Dirty sand and gravel of restricted permeability (GM, GM-GP, SM, SM-SP)	77
Stiff residual silts and clays, silty fine sands, clayey sands and gravels (CL, ML, CH, MH, SM, SC, GC)	82
Very soft to soft clay, silty clay, organic silt and clay (CL, ML, OL, CH, MH, OH)	106
Medium to stiff clay deposited in chunks and protected from infiltration (CL, CH)	142

*Soil types are based on USDA Unified Soil Classification System; see Table 4-4 for soil type definitions.

Table 4-4. Soil Type Definitions Based on USDA Unified Soil Classification System

Soil Type	Group Symbol	Description
Gravels	GW	Well-graded gravels and gravel mixtures
	GP	Poorly graded gravel-sand-silt mixtures
	GM	Silty gravels, gravel-sand-silt mixtures
	GC	Clayey gravels, gravel-sand-clay mixtures
Sands	SW	Well-graded sands and gravelly sands
	SP	Poorly graded sands and gravelly sands
	SM	Silty sands, poorly graded sand-silt-mixtures
	SC	Clayey sands, poorly graded sand-clay mixtures
Fine Grain Silt and Clays	ML	Inorganic silts and clayey silts
	CL	Inorganic clays of low to medium plasticity
	OL	Organic silts and organic silty clays of low plasticity
	MH	Inorganic silts, micaceous or fine sands or silts, elastic silts
	CH	Inorganic clays of high plasticity, fine clays
	OH	Organic clays of medium to high plasticity

4.1.2.5 Combined Saturated Soil and Water Forces

When a structure is subject to hydrostatic forces from both saturated soil and standing water (illustrated in Figure 4-8), the resultant combined lateral force, f_{comb}, is the sum of the lateral water hydrostatic force, f_{sta}, and the differential between the water and soil pressures, f_{dif}. The basic equation for computing f_{dif} is shown in Equation 4-5.

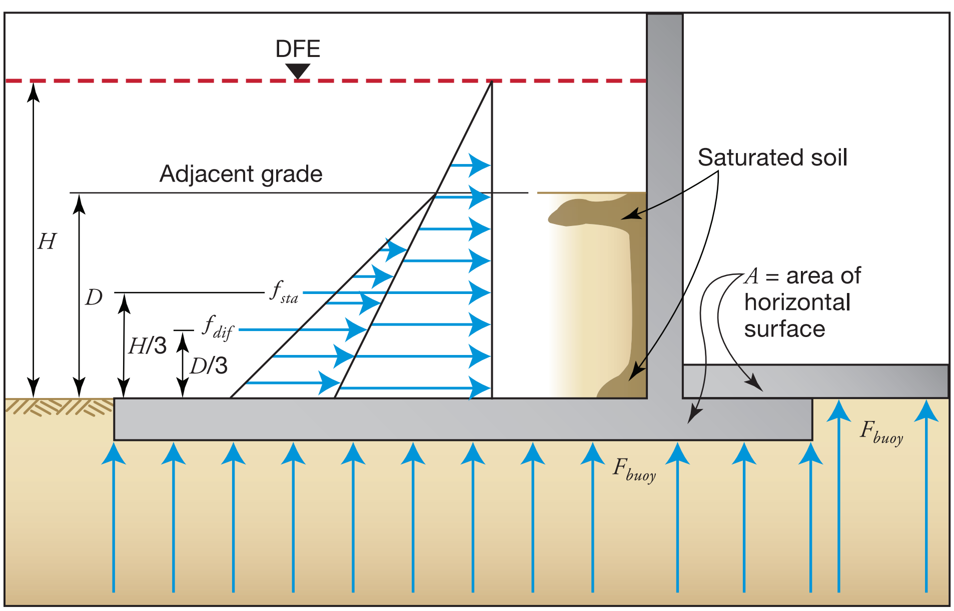


Figure 4-8. Combination soil/water hydrostatic and buoyancy forces

EQUATION 4-5: SUBMERGED SOIL AND WATER FORCES

$$f_{dif} = \frac{1}{2}(S - \gamma_w)D^2 \qquad \text{(Eq. 4-5)}$$

where:

f_{dif} = differential soil/water force acting at a distance D/3 from the point under consideration (lb/lf)

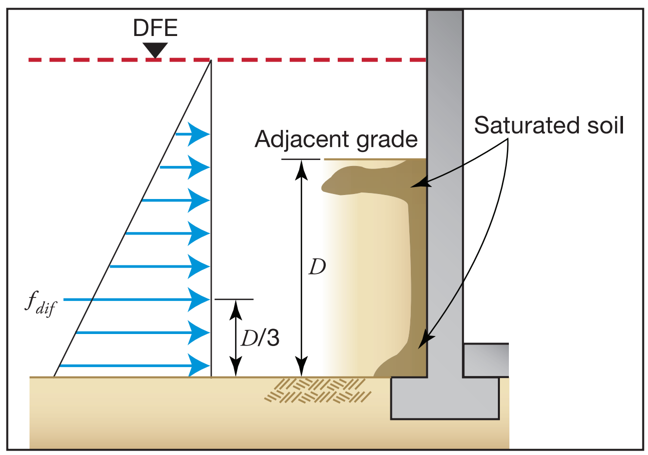
S = equivalent fluid weight of submerged soil and water (lb/ft^3) as shown in Table 4-3

D = depth of saturated soil from adjacent grade to the top of the footer (ft)

γ_w = specific weight of water (62.4 lb/ft^3 for fresh water and 64.0 lb/ft^3 for saltwater)

NOTE

f_{dif} acts at a point D/3 where D is the distance from the adjacent grade to the top of the foundation footer.



4.1.2.6 Vertical Hydrostatic Forces

The basic equation for analyzing the vertical hydrostatic force (buoyancy) due to standing water (illustrated by Figure 4-7) is shown in Equation 4-6.

The computation of hydrostatic forces is vital to the successful design of floodwalls, sealants, closures, shields, foundation walls, and a variety of other retrofitting measures. The Hydrostatic Force Computation Worksheet (Figure 4-9) can be used to conduct hydrostatic calculations.

EQUATION 4-6: BUOYANCY FORCES

$$F_{buoy} = \gamma_w (Vol) \quad \text{(Eq. 4-6)}$$

where:

F_{buoy} = vertical hydrostatic force resulting from the displacement of a given volume of floodwater (lb)

γ_w = specific weight of water (62.4 lb/ft^3 for fresh water and 64.0 lb/ft^3 for saltwater)

Vol = volume of floodwater displaced by a submerged object (ft^3)

Hydrostatic Force Computation Worksheet

Owner Name:____________________ Prepared By:____________________

Address: ______________________________ Date: ______________

Property Location: ____

Constants

γ_w = specific weight of water = 62.4 lb/ft^3 for fresh water and 64.0 lb/ft^3 for saltwater

Variables

H = floodproofing design depth (ft) =

D = depth of saturated soil (ft) =

S = equivalent fluid weight of saturated soil (lb/ft^3) =

Vol = volume of floodwater displaced by a submerged object (ft^3) =

Summary of Loads

f_{sta} =

f_{dif} =

f_{comb} =

F_{bouy} =

Lateral Hydrostatic Force (see Equation 4-4)

$$f_{sta} = \frac{1}{2} P_h H = \frac{1}{2} \gamma_w H^2$$

Submerged Soil and Water Forces (see Equation 4-5)

$$f_{dif} = \frac{1}{2}(S - \gamma_w) D^2$$

Buoyancy Force (see Equation 4-6)

$$F_{buoy} = \gamma_w (Vol)$$

Note: Equations 4-4 and 4-5 do not account for equivalent hydrostatic loads due to the low velocity of floodwaters (less than 10 ft/sec). If velocity floodwater exists, use Equations 4-7 and 4-8. Refer to Chapter 8 of FEMA P-55, *Coastal Construction Manual* (FEMA, 2011) for discussion of hydrostatic loads.

Figure 4-9. Hydrostatic Force Computation Worksheet

4.1.2.7 Hydrodynamic Forces

When floodwater flows around a structure, it imposes additional loads on the structure, as shown in Figure 4-10. These loads are a function of flow velocity and structural geometry.

Low velocity hydrodynamic forces are defined as situations where floodwater velocities do not exceed 10 ft/sec, while high velocity hydrodynamic forces involve floodwater velocities in excess of 10 ft/sec.

NOTE

Hydrodynamic forces have been shown to act on slab-on-grade houses to maximize their effects.

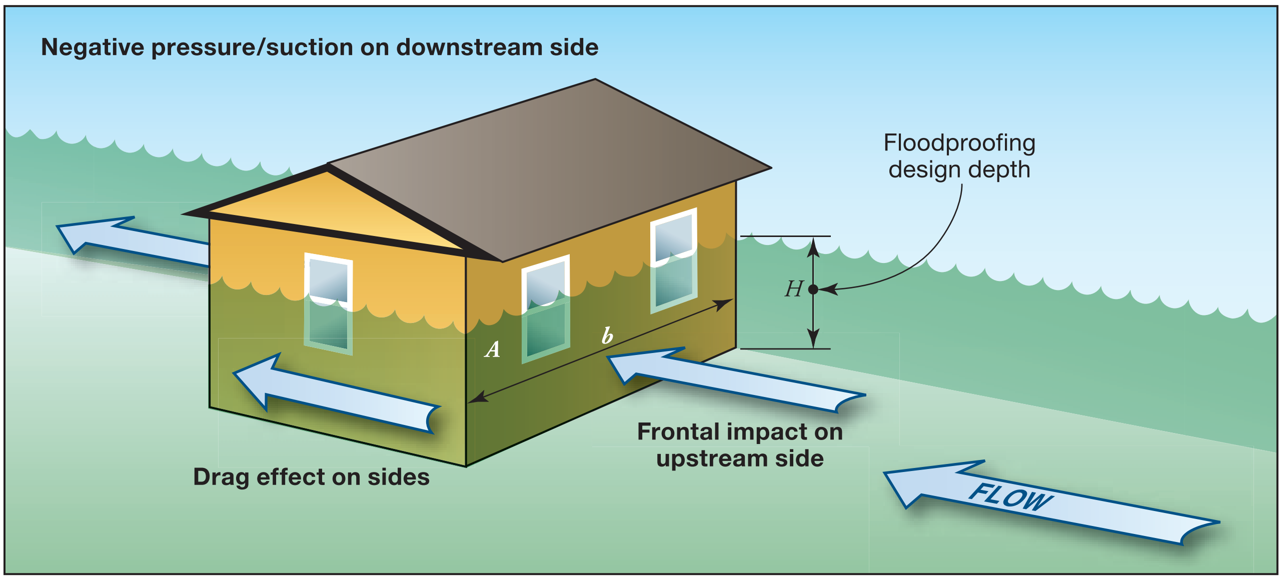


Figure 4-10. Hydrodynamic and impact forces

Low Velocity Hydrodynamic Forces

In cases where velocities do not exceed 10 ft/sec, the hydrodynamic effects of moving water can be converted to an equivalent hydrostatic force by increasing the depth of the water (head) above the flood level by an amount *dh*, which is shown in Equation 4-7.

The drag coefficient used in Equation 4-7 is taken from the *Shore Protection Manual, Volume 2* (USACE, 1984) and additional guidance is provided in ASCE 7. The drag coefficient is a function of the shape of the object around which flow is directed. The value of C_d, unless otherwise evaluated, shall not be less than 1.25 and can be determined from the width-to-height ratio, b/H, of the structure in question. The width *(b)* is the length of the side perpendicular to the flow, and the height *(H)* is the distance from the floodproofing design depth to the LAG level. Table 4-5 gives C_d values for different width-to-height ratios.

NOTE

Sources of data for determining flood flow velocity include hydraulic calculations, historical measurements, and rules of thumb. Floodwater that is 1 foot deep moving in excess of 5 ft/sec can knock an adult over and cause erosion of stream banks. Overbank velocities are usually less than stream channel velocities. If no data for flood flow velocity exists for a site, the reader should contact an experienced hydrologist or hydraulic engineer for estimates.

EQUATION 4-7: CONVERSION OF LOW VELOCITY FLOW TO EQUIVALENT HEAD

$$dh = \frac{C_d V^2}{2g} \qquad \text{(Eq. 4-7)}$$

where:

dh = equivalent head due to low velocity flood flows (ft)

C_d = drag coefficient (from Table 4-5)

V = velocity of floodwater (ft/sec)

g = acceleration of gravity (equal to 32.2 ft/sec^2)

Table 4-5. Drag Coefficients for Ratios of Width to Height (w/h)

Width to Height Ratio *(b/H)*	Drag Coefficient *(C_d)*
1–12	1.25
13–20	1.3
21–32	1.4
33–40	1.5
41–80	1.75
81–120	1.8
>120	2.0

The value *dh* is then converted to an equivalent hydrostatic pressure through use of the basic equation for lateral hydrostatic forces introduced earlier in this chapter and modified, as shown in Equation 4-8.

EQUATION 4-8: CONVERSION OF EQUIVALENT HEAD TO EQUIVALENT HYDROSTATIC FORCE

$$f_{dh} = \gamma_w (dh) H = P_{dh} H \qquad \text{(Eq. 4-8)}$$

where:

f_{dh} = equivalent hydrostatic force due to low velocity flood flows (lb/lf)

γ_w = specific weight of water (62.4 lb/ft^3 for fresh water and 64.0 lb/ft^3 for saltwater)

dh = equivalent head due to low velocity flood flows (ft)

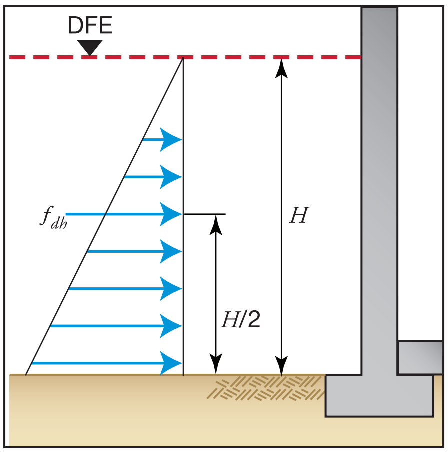
H = floodproofing design depth (ft)

P_{dh} = hydrostatic pressure due to low velocity flood flows (lb/ft^2) *($P_{dh} = \gamma_w(dh)$)*

EQUATION 4-8: CONVERSION OF EQUIVALENT HEAD TO EQUIVALENT HYDROSTATIC FORCE

(concluded)

NOTE



Although f_{dh} is considered a hydrostatic force for velocities under 10 ft/sec, it acts at a point $H/2$, similarly to lateral hydrodynamic forces.

Equivalent Hydrostatic Force Computation Worksheet

Owner Name:____________________ Prepared By:____________________

Address: ______________________________ Date: ________________

Property Location: ____

Constants

γ_w = specific weight of water = 62.4 lb/ft^3 for fresh water and 64.0 lb/ft^3 for saltwater

g = acceleration of gravity = 32.2 ft/sec^2

Variables

H = design floodproof depth (ft) =

V = velocity of floodwater (10 ft/sec or less) =

P_{dh} = hydrostatic pressure due to low velocity flood flows (lb/ft^2) =

b = width of structure perpendicular to flow (ft) =

Summary of Loads

f_{dh} =

f_{sta} =

f_{dif} =

f_{comb} =

Conversion of Low Velocity Flood Flow to Equivalent Head (see Equation 4-7)

$$f_{dh} = \gamma_w (dh) H = P_{dh} H$$

Develop C_d:

b/H =

From Table 4-5; C_d =

Conversion of Equivalent Head to Equivalent Hydrostatic Force (see Equation 4-8)

$$P_d = C_d \rho \frac{V^2}{2}$$

Figure 4-11. Equivalent Hydrostatic Force Computation Worksheet

4.1.2.8 High Velocity Hydrodynamic Forces

For special structures and conditions, and for velocities greater than 10 ft/sec, a more detailed analysis and evaluation should be made utilizing basic concepts of fluid mechanics and/or hydraulic models. The basic equation for hydrodynamic pressure is shown in Equation 4-9.

EQUATION 4-9: HIGH VELOCITY HYDRODYNAMIC PRESSURE

$$P_d = C_d \rho \frac{V^2}{2} \qquad \text{(Eq. 4-9)}$$

where:

P_d = hydrodynamic pressure (lb/ft^2)

C_d = drag coefficient (taken from Table 4-5)

ρ = mass density of fluid (1.94 $slugs/ft^3$ for fresh water and 1.99 $slugs/ft^3$ for saltwater)

V = velocity of floodwater (ft/sec)

After determination of the hydrodynamic pressure (P_d), the total force (F_d) against the structure (see Figure 4-10) can be computed as the pressure times the area over which the water is affecting (see Equation 4-10).

EQUATION 4-10: TOTAL HYDRODYNAMIC FORCE

$$F_d = P_d A \qquad \text{(Eq. 4-10)}$$

where:

F_d = total force against the structure (lb)

P_d = hydrodynamic pressure (lb/ft^2)

A = submerged area of the upstream face of the structure (ft^2)

Figure 4-12 can be used in the computation of high velocity hydrodynamic forces.

Hydrodynamic Force (High Velocity) Computation Worksheet	
Owner Name:__________ Prepared By:__________	
Address: __________ Date: __________	
Property Location: __________	
Constants ρ = mass density of fluid (1.94 slugs/ft³ for fresh water and 1.99 slugs/ft³ for saltwater) **Variables** V = velocity of floodwater, >10 ft/sec = C_d = drag coefficient = A = submerged area of upstream face of structure (ft²) =	**Summary of Loads** P_d = F_d =
High Velocity Hydrodynamic Pressure (see Equation 4-9)	
$P_d = C_d \rho \frac{V^2}{2}$ Develop C_d: b/H = From Table 4-5; C_d =	
Total Hydrodynamic Force (see Equation 4-10)	
$F_d = P_d A$	

Figure 4-12. Hydrodynamic Force (High Velocity) Computation Worksheet

4.1.2.9 Impact Loads

Impact loads are imposed on the structure by objects carried by the moving water. The magnitude of these loads is very difficult to predict, but some reasonable allowance must be made for them in the design of retrofitting measures for potentially affected buildings. To arrive at a realistic allowance, considerable judgment must be used, along with the designer's knowledge of debris problems at the site and consideration of the degree of exposure of the structure. Impact loads are classified as:

CROSS REFERENCE

Section 5.4.5 of ASCE 7-10 and the corresponding commentary contain an extensive discussion on computing riverine and coastal impact loads.

- no impact (for areas of little or no velocity or potential source of debris);
- normal impact;
- special impact; and
- extreme impact.

Normal Impact Forces

Normal impact forces relate to isolated occurrences of typically sized debris or floating objects striking the structure (see Figure 4-10 for location of frontal impact from debris). For design purposes, this can be considered a concentrated load acting horizontally at the flood elevation, or any point below it, equal to the impact force created by a typical object traveling at the velocity of the floodwater acting on a 1-square-foot surface of the submerged structure area perpendicular to the flow. Typical object size and mass will vary by location, but ASCE 7-10 *Commentary,* section C5.4.5 (Debris Weight), provides some guidance. The calculation of normal impact forces (loads) is shown in Equation 4-11.

The equation for calculating debris loads is given in the ASCE 7, *Commentary*. The equation has been converted into Equation 4-11, based on assumptions appropriate for the typical structures that are covered in this document.

NOTE

The assumption that debris velocity is equal to the flood velocity may overstate the velocities of large debris objects; therefore, engineering judgment may be required in some instances. Designers may wish to reduce debris velocity for larger objects.

EQUATION 4-11: NORMAL IMPACT LOADS

$$F_i = W\,V\,C_D C_B C_{Str} \qquad \text{(Eq. 4-11)}$$

where:

F_i = impact force acting at the BFE (lb)

W = weight of the object (lb)

V = velocity of water (ft/sec)

C_D = depth coefficient (see Table 4-6)

C_B = blockage coefficient (taken as 1.0 for no upstream screening, flow path greater than 30 ft; see Table 4-7 for more information)

C_{Str} = building structure coefficient

= 0.2 for timber pile and masonry column supported structures 3 stories or less in height above grade

= 0.4 for concrete pile or concrete or steel moment resisting frames 3 stories or less in height above grade

= 0.8 for reinforced concrete (including insulated concrete) and reinforced masonry foundation walls

Table 4-6. Depth Coefficient (C_D) by Flood Hazard Zone and Water Depth

Flood Hazard Zone and Water Depth	C_D
Floodway[1] or Zone V	1.0
Zone A, stillwater flood depth > 5 ft	1.0
Zone A, stillwater flood depth = 4 ft	0.75
Zone A, stillwater flood depth = 3 ft	0.5
Zone A, stillwater flood depth = 2 ft	0.25
Zone A, stillwater flood depth < 1 ft	0.0

1 Per ASCE 24, a "floodway" is a "channel and that portion of the floodplain reserved to convey the base flood without cumulatively increasing the water surface elevation more than a designated height."

Table 4-7. Values of Blockage Coefficient (C_B)

Degree of Screening or Sheltering within 100 ft Upstream	C_B
No upstream screening, flow path wider than 30 ft	1.0
Limited upstream screening, flow path 20 ft wide	0.6
Moderate upstream screening, flow path 10 ft wide	0.2
Dense upstream screening, flow path less than 5 ft wide	0.0

Often, there are regional differences between the size, shape, and weight of water-borne debris, and the debris velocity. Designers should consider locally adopted guidance because it may be based on more recent information or information specific to the local hazards than the information in ASCE 7.

The parameters in Equation 4-11 are also discussed in detail in Chapter 8 of FEMA P-55, *Coastal Construction Manual* (FEMA, 2011, Fourth Edition).

Special and Extreme Impact Forces

Special impact forces occur when large objects or conglomerates of floating objects, such as ice floes or accumulations of floating debris, strike a structure. Where stable natural or artificial barriers exist that would effectively prevent these special impact forces from occurring, these forces may not need to be considered in the design. Details for calculating special impact loads are outlined in the ASCE 7 commentary section C5.4.5.

NOTE

Where extreme impact loads are a threat, the preferred retrofitting alternative is relocation.

Extreme impact forces occur when large, floating objects, such as runaway barges or collapsed buildings and structures, strike the structure (or a component of the structure). These forces generally occur within the floodway or areas of the floodplain that experience the highest velocity flows. It is impractical to design residential buildings to have adequate strength to resist extreme impact forces.

Impact forces are critical design considerations that must be thoroughly evaluated. The following Impact Force Computation Worksheet, Figure 4-13, can be used to conduct normal impact force calculations.

Impact Force Computation Worksheet	
Owner Name:______________________	Prepared By:______________________
Address: ______________________	Date: ______________
Property Location: ______________________	

Variables

W = weight of the object (lb)

V = velocity of water (ft/sec)

C_D = depth coefficient (see Table 4-6)

C_B = blockage coefficient (taken as 1.0 for no upstream screening, flow path greater than 30 ft; see Table 4-7 for more information)

C_{Str} = building structure coefficient

= 0.2 for timber pile and masonry column supported structures 3 stories or less in height above grade

= 0.4 for concrete pile or concrete or steel moment resisting frames 3 stories or less in height above grade

= 0.8 for reinforced concrete foundation walls (including insulated concrete forms)

Summary of Loads

F_i =

Normal Impact Loads (see Equation 4-11)

$$F_i = W V C_D C_B C_{Str}$$

Figure 4-13. Impact Force Computation Worksheet

4.1.2.10 Riverine Erosion

The analysis of erosion that impacts stream banks and nearby overbank structures is a detailed effort that is usually accompanied by detailed geotechnical investigations. Some of the variables that impact the stability (or erodibility) of the stream banks include the following:

- critical height of the slope;
- inclination of the slope;
- cohesive strength of the soil in the slope;
- distance of the structure in question from the shoulder of the stream bank;
- degree of stabilization of the surface of the slope;

CROSS REFERENCE

1- to 24-hour rainfall intensities for each State are available from the National Weather Service (NWS) *Technical Memorandum NWS HYDRO-35,* “Technical Paper 40, NOAA Atlas 2” or “NOAA Atlas 14” available for download at: http://www.nws.noaa.gov/oh/hdsc/currentpf.htm.

Rainfall intensities are available for a range of storm frequencies, including the 2-, 10-, 25-, 50-, and 100-year events. The 2- or 10-year intensity rainfall is considered a minimum design value for pumping rates when floodwater prevents gravity discharge from floodwalls and levees. The 100-year intensity rainfall should be the maximum design value for sizing gravity flow pipes and/or closures.

- level and variation of groundwater within the slope;
- level and variation in level of water on the toe of the slope;
- tractive shear stress of the soil; and
- frequency of rise and fall of the surface of the stream.

Both FEMA and the USACE have researched the stability of stream banks in an effort to quantify stream bank erosion. However, concerns over the universal applicability of the research results preclude their inclusion in this manual. It is suggested that, when dealing with stream banks susceptible to erosion, the designer contact a qualified geotechnical engineer or a hydraulic engineer experienced in channel stability.

4.1.3 Site Drainage

The drainage system for the area enclosed by a floodwall or levee must accommodate the precipitation runoff from this interior area (and any contributing areas such as roofs and higher ground parcels) and the anticipated seepage through or under the floodwall or levee during flooding conditions.

> **NOTE**
>
> The rational equation $Q = ci_rA$ is used to compute the amount of precipitation runoff from small areas. It is generally not applicable to drainage areas greater than 10 acres in size.

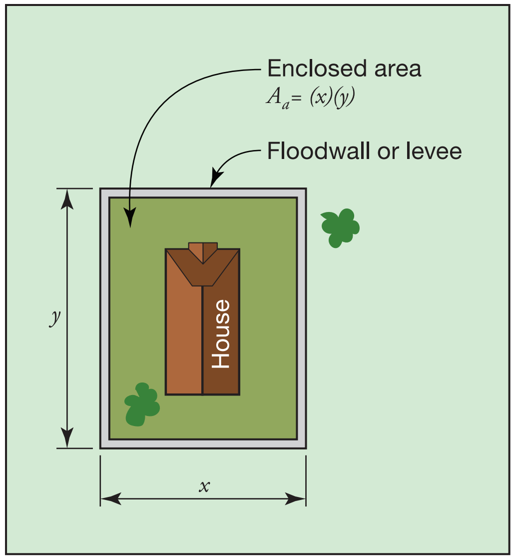
There are two general methods for removing interior drainage. The first is a gravity flow system, which provides a means for interior drainage of the protected area when there is no floodwater against the floodwall or levee. This is accomplished by placing a pipe(s) through the floodwall or levee with a flap gate attachment. The flap gate prevents flow from entering the interior area through the drainpipe when floodwater rises above the elevation of the pipe.

The second method, a pump system, removes accumulation of water when the elevation of the floodwater exceeds the elevation of the gravity drain system. A collection system composed of pervious trenches, underground tiles, or sloped surface areas transports the accumulating water to a sump area. In the levee application, these drains should be incorporated into the collection system. The anticipated seepage from under and through floodwalls and levees must also be taken into consideration by combining it with flow from precipitation (see Figure 4-14). It is important to verify that the pump system has a reliable power source that can handle the flooding in the area enclosed by the floodwall or levee. This is essential to the performance of the floodwall or levee system.

> **NOTE**
>
> The residential terrain runoff coefficient, c, is used to model the runoff characteristics of various land uses. Use the value for the predominant land use within a specific area or develop a weighted average for areas with multiple land uses. The most common coefficients are 0.70 for residential areas, 0.90, for commercial areas, and 0.40 for undeveloped land.

To determine the amount of precipitation that can collect in the contained area, the rainfall intensity, given in inches per hour, must be determined for a particular location. This value is multiplied by the enclosed area, A_a, in square feet, a residential terrain runoff coefficient *(c)* of 0.7, and a conversion factor of 0.01. The answer is given in gallons per minute (gpm). See Equation 4-12.

Figure 4-14. Rectangular area enclosed by a floodwall or levee



EQUATION 4-12: RUNOFF QUANTITY IN AN ENCLOSED AREA

$$Q_a = 0.01 c i_r A_a \qquad \text{(Eq. 4-12)}$$

where:

Q_a = runoff from the enclosed area (gpm)

0.01 = factor converting the answer to gpm

c = residential terrain runoff coefficient of 0.7

i_r = intensity of rainfall (in./hr)

A_a = is the area enclosed by the floodwall or levee (ft^2)

NOTE

When determining the minimum discharge size for pumps within enclosed areas, the designer should consider the impacts of lag time between storms that control the gravity flow mechanism (i.e., inside and outside the enclosed area) and the storage capacity within the enclosed area after the gravity discharge system closes. If the designer is not familiar with storm lag time and the computation of storage within an enclosed area, an experienced hydrologist or hydraulic engineer should be consulted.

In some cases, a levee or floodwall may extend only partially around the property and tie into higher ground (see Figure 4-15). For such cases, the amount of precipitation that can flow downhill as runoff into the protected area, A_a, must be included. To calculate this value, the additional area of land, A_b, that can discharge water into the enclosure should be estimated. This value is then multiplied by the previously determined rainfall intensity, i_r, by the most suitable terrain coefficient, and by 0.01. See Equation 4-13.

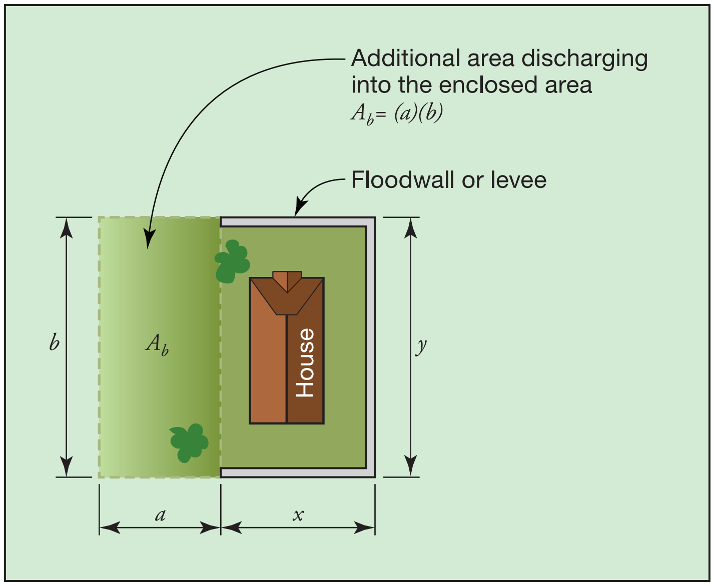


Figure 4-15. Rectangular area partially enclosed by a floodwall or levee

EQUATION 4-13: RUNOFF QUANTITY FROM HIGHER GROUND INTO A PARTIALLY ENCLOSED AREA

$$Q_b = 0.01 c i_r A_b \qquad \text{(Eq. 4-13)}$$

where:

Q_b = runoff from additional contributing area (gpm)

0.01 = factor converting the answer to gpm

c = most suitable terrain runoff coefficient

i_r = is the intensity of rainfall (in./hr)

A_b = area discharging to the area partially enclosed by the floodwall or levee (ft^2)

Seepage flow rates from the levee or floodwall, Q_c, must also be estimated. In general, unless the seepage rate is calculated by a qualified soils engineer, a value of 2 gpm for every 300 feet of levee or 1 gpm for every 300 feet of floodwall should be assumed during base 100-year-flood conditions. See Equation 4-14.

EQUATION 4-14: SEEPAGE FLOW RATE THROUGH A FLOODWALL OR LEVEE

$$Q_c = sr(l) \qquad \text{(Eq. 4-14)}$$

where:

Q_c = seepage rate through the floodwall/levee (gpm)

sr = seepage rate (gpm) per foot of floodwall/levee

l = length of the floodwall/levee (ft)

The values for inflow within the enclosed area, runoff from uphill areas draining into the enclosure, and seepage through the floodwall/levee should be added together to obtain the minimum discharge size, Q_{sp}, in gpm for the pump. See Equation 4-15.

EQUATION 4-15: MINIMUM DISCHARGE FOR PUMP INSTALLATION

$$Q_{sp} = Q_a + Q_b + Q_c \qquad \text{(Eq. 4-15)}$$

where:

Q_{sp} = minimum discharge for pump installation (gpm)

Q_a = discharge from an enclosed area (from Equation 4-14) (gpm)

Q_b = discharge from higher ground to partially enclosed area (from Equation 4-15) (gpm)

Q_c = discharge from seepage through a floodwall or levee (from Equation 4-16) (gpm)

Important considerations in determining the minimum discharge size of a pump include storage available within the enclosed area and the lag time between storms that impact the enclosed area and the area to which the enclosed area drains. Pumps will continue to operate during flooding events (assuming power is constant or backup power is available), but gravity drains will close once the floodwater elevation outside of the enclosed area exceeds the elevation of the drain pipe/flap gate. Therefore, the critical design issue is to determine runoff and seepage that occurs once the flap gate closes. Typical design solutions incorporate a freeboard of several inches or more to safely control the 10-year flood event.

Figure 4-16 can be used to calculate the minimum discharge for pump installations.

Interior Drainage Computation Worksheet
Owner Name:________________ Prepared By:________________
Address: ________________ Date: ________________
Property Location: ________________

Constants

0.01 = factor converting the answer to gpm

Variables

A_a = is the area enclosed by the floodwall or levee (ft^2)

A_b = area discharging to the area partially enclosed by the floodwall or levee (ft^2)

c = residential terrain runoff coefficient of 0.7

i_r = intensity of rainfall (in./hr)

sr = seepage rate (gpm) per foot of floodwall/levee

l = length of the floodwall/levee (ft)

Summary of Loads

Q_{sp} =

Q_a =

Q_b =

Q_c =

Runoff Quantity in an Enclosed Area (see Equation 4-12)
$Q_a = 0.01ci_rA_a$
Runoff Quantity From Higher Ground into a Partially Enclosed Area (see Equation 4-13)
$Q_b = 0.01ci_rA_b$
Seepage Flow Rate Through a Levee or Floodwall (see Equation 4-14)
$Q_c = sr(l)$
Minimum Discharge for Pump Installation (see Equation 4-15)
$Q_{sp} = Q_a + Q_b + Q_c$

Figure 4-16. Interior Drainage Computation Worksheet

4.1.3.1 Closed Basin Lakes

Two types of lakes pose special hazards to adjacent development: lakes with no outlets, such as the Great Salt Lake and the Salton Sea (California); and lakes with inadequate or elevated outlets, such as the Great Lakes and many glacial lakes. These two types are referred to as "closed basin lakes." Closed basin lakes are subject to very large fluctuations in elevation and can retain persistent high water levels.

Closed basin lakes occur in almost every part of the United States for a variety of reasons: lakes in the northern tier of States and Alaska were scoured out by glaciers; lakes with no outlets (playas) formed in the west due to tectonic action; oxbow lakes along the Mississippi and other large rivers formed as a result of channel migration; and sinkhole lakes form in areas with large limestone deposits at or near the surface where there is adequate surface water and rainfall to dissolve the limestone (Karst topography).

CROSS REFERENCE

More information on closed basin lakes, alluvial fan, and movable bed stream hazards can be obtained from the *National Flood Insurance Program Community Rating System, Special Hazards Supplement to the CRS Coordinator's Manual* (FEMA, 2006b).

Determination of the flood elevations for closed basin lakes follows generally accepted hydrological methods, which incorporate statistical data, historical high water mark determinations, stage-frequency analysis, topographical analysis, water balance analysis, and combinations of these methods. The flood-prone area around a closed basin lake is referred to in affected DFIRM panels as an Area of Special Consideration (ASC). The ASC may include the 1- and 0.2-percent-annual-chance floodplains and additional areas to account for the continuous and often uncertain fluctuations in the water-surface elevation due to the closed-basin lake phenomenon. The ASC is an area subject to flooding, but the percent chance of being flooded in any given year is not defined.

4.1.4 Movable Bed Streams

Erosion and sedimentation are factors in the delineation and regulation of almost all riverine floodplains. In many rivers and streams, these processes are relatively predictable and steady. In other streams, sedimentation and erosion are continual processes, often having a larger impact on the extent of flooding and flood damages than the peak flow.

Extreme cases of sedimentation and erosion are a result of both natural and engineered processes. They frequently occur in the arid west, where relatively recent tectonic activity has left steep slopes, rainfall and streamflow are infrequent, and recent and rapid development has disturbed the natural processes of sediment production and transport.

Movable bed streams include streams where erosion (degradation of the streambed), sedimentation (aggradation of the streambed), or channel migration cause a change in the topography of the stream sufficient to change the flood elevation or the delineation of the floodplain or floodway. Analysis of movable bed streams generally includes a study of the sources of sediment, changes in those sources, and the impact of sediment transport through the floodplain.

4.1.5 Analysis of Non-Flood-Related Hazards

While floods continue to be a major hazard to homes nationwide, they are not the only natural hazard that causes damage to residential buildings. Parts of the United States are subject to high winds that can accompany thunderstorms, hurricanes, tornadoes, and frontal passages. In addition, many regions are threatened by earthquake fault areas, land subsidence, and fire and snow hazards (Figure 4-17).

Retrofitting measures can be designed to modify structures to reduce the chance of damage from wind and other non-flood-related hazards. Fortunately, strengthening a home to resist earthquake damage can also increase its ability to withstand wind damage and flood-related impact and velocity forces.

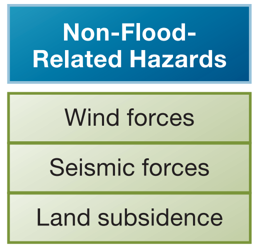


Figure 4-17. Non-flood-related natural hazards

4.1.6 Wind Forces

High winds impose significant forces on a home and the structural elements of its foundation. Damage potential is increased when the wind forces occur in combination with flood forces, often in coastal areas. In addition, as a structure is elevated to minimize the effects of flood forces, the wind loads on the elevated structure may be increased, depending on the amount of elevation and the structure's exposure to wind forces.

> **NOTE**
>
> The designer must be aware that retrofitting actions may trigger a threat from multiple natural hazards and be prepared to address these issues.

Wind forces exert pressure on structural components such as walls, roofs, connections, and foundations. Therefore, wind loads should be considered in the design process at the same time as hydrostatic, hydrodynamic, impact, and building dead and live loads, and loads from other natural hazards such as earthquakes.

> **CROSS REFERENCE**
>
> Refer to FEMA P-55, *Coastal Construction Manual* (FEMA, 2011), and ASCE 7 for a detailed discussion of wind forces.

A detailed discussion for computation of wind forces is beyond the scope of this publication. However, FEMA P-55, *Coastal Construction Manual*, (FEMA, 2011) provides details on the basic parameters for determining wind loads:

- basic wind speed (see ASCE 7 or IRC wind speed map, V);
- wind directionality factor, K_d (see ASCE 7);
- building exposure category, B, C, or D (see ASCE 7);
- topographic factor, K_{zt} (see ASCE 7);
- gust effect factor, typically 0.85 (see ASCE 7);
- enclosure classification, open, partially enclosed, or enclosed (see ASCE 7); and
- internal pressure coefficient, GC_{pi} (see ASCE 7).

When wind interacts with a building, both positive and negative pressures simultaneously occur (see Figure 4-18). To prevent wind induced building failure, buildings must have sufficient strength to resist the applied loads from these pressures. As previously mentioned, the magnitude of pressure is a function of several primary factors: exposure, basic wind speed, topography, building height, building shape, and internal

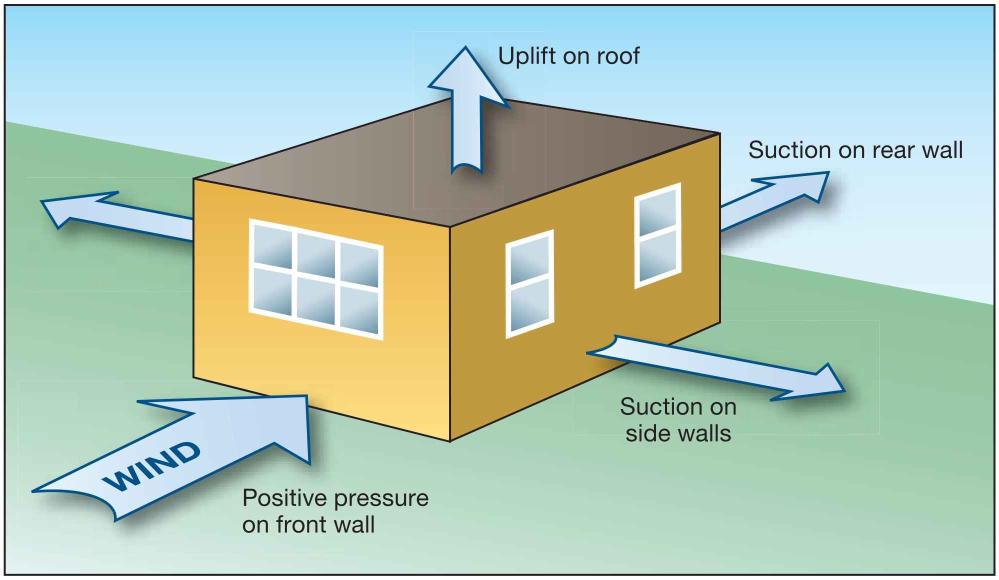


Figure 4-18. Wind-induced pressures on a building

pressure classification. Once these parameters are defined, the engineer can determine the design pressures, and apply these pressures to the appropriate tributary area for the element or connection to be analyzed.

The concept of wind producing significant forces on a structure is based on the velocity difference of a medium (air) striking an obstruction (the structure). Wind speeds vary, depending on the location within the United States and the frequency with which these loads occur. ASCE 7 and the IRC provide basic wind speed maps showing these wind velocities and frequencies. The design velocity for a particular site can be determined from these maps. If the local code enforced is the IRC, the designer should refer to the IRC wind speed maps (Figures 4-19 A and B). If no local code is in force, the designer should refer to ASCE 7, *Minimum Design Loads for Buildings and Other Structures*.

FEMA has completed several building performance assessments following Hurricanes, including Andrew (1992), Iniki (1992), Opal (1995), Fran (1996), Georges (1998), Ivan (2004), Charley (2004), Katrina (2005), and Ike (2008). FEMA assessed the structural performance of residential building systems damaged by hurricane winds; provided findings and recommendations for enhancing building performance under hurricane wind conditions; and addressed building materials, code compliance, plan review, construction techniques, quality of construction, and construction inspection issues.

CROSS REFERENCE

Copies of the building performance assessment reports can be obtained from the FEMA library: http://www.fema.gov/library

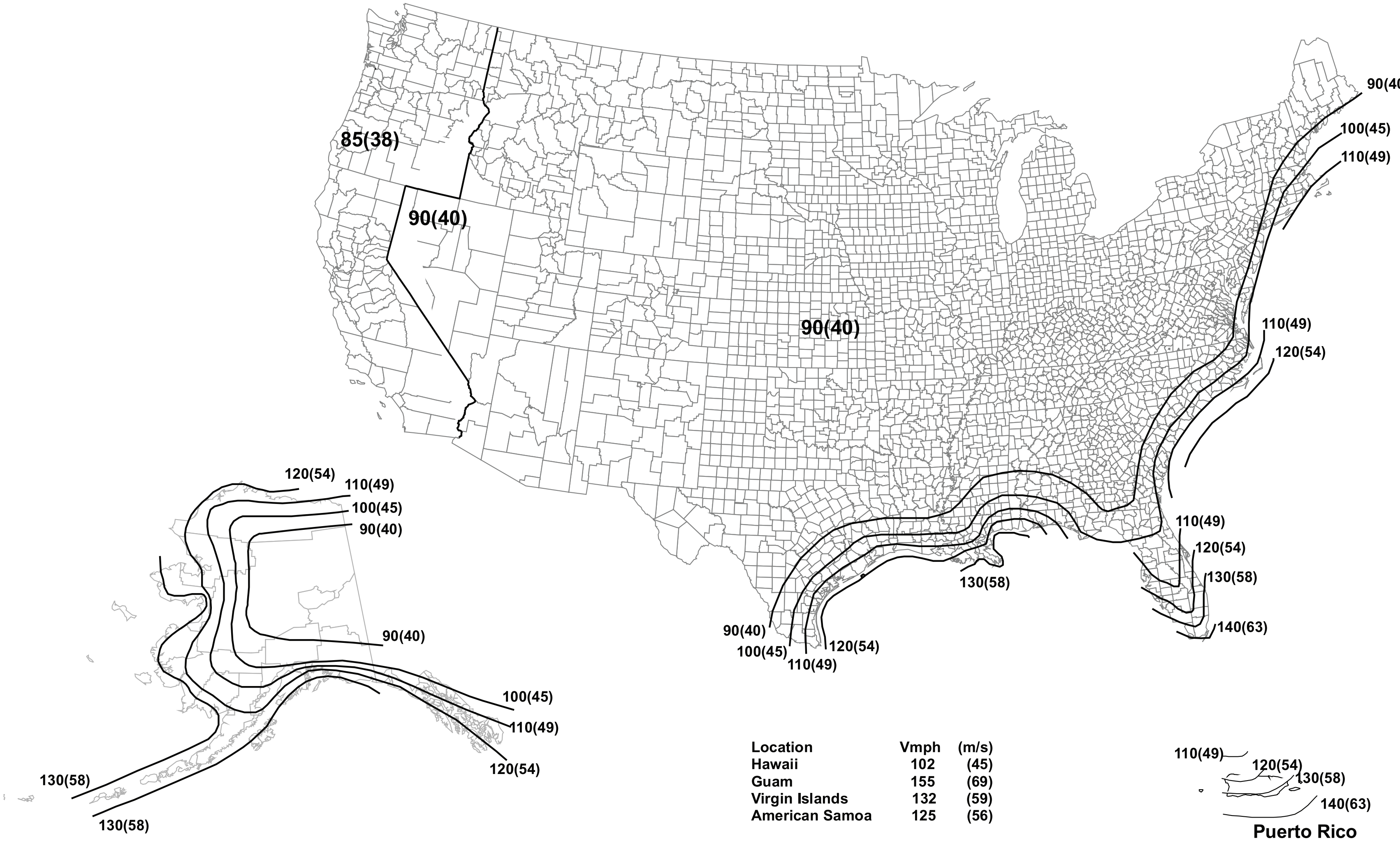
FEMA 488, *Hurricane Charley in Florida – Observations, Recommendations, and Technical Guidance*, 2005

FEMA 489, *Hurricane Ivan in Alabama and Florida – Observations, Recommendations, and Technical Guidance*, 2005

FEMA 549, *Hurricane Katrina in the Gulf Coast – Building Performance Observations, Recommendations, and Technical Guidance*, 2006

FEMA P-757, *Hurricane Ike in Texas and Louisiana – Building Performance Observations, Recommendations, and Technical Guidance*, 2009

FEMA P-765, *Midwest Floods of 2008 in Iowa and Wisconsin – Building Performance Observations, Recommendations, and Technical Guidance*, 2009



Location	Vmph	(m/s)
Hawaii	102	(45)
Guam	155	(69)
Virgin Islands	132	(59)
American Samoa	125	(56)

Notes:
1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10 m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

Figure 4-19A. Basic wind speed map

SOURCE: IRC, USED WITH PERMISSION

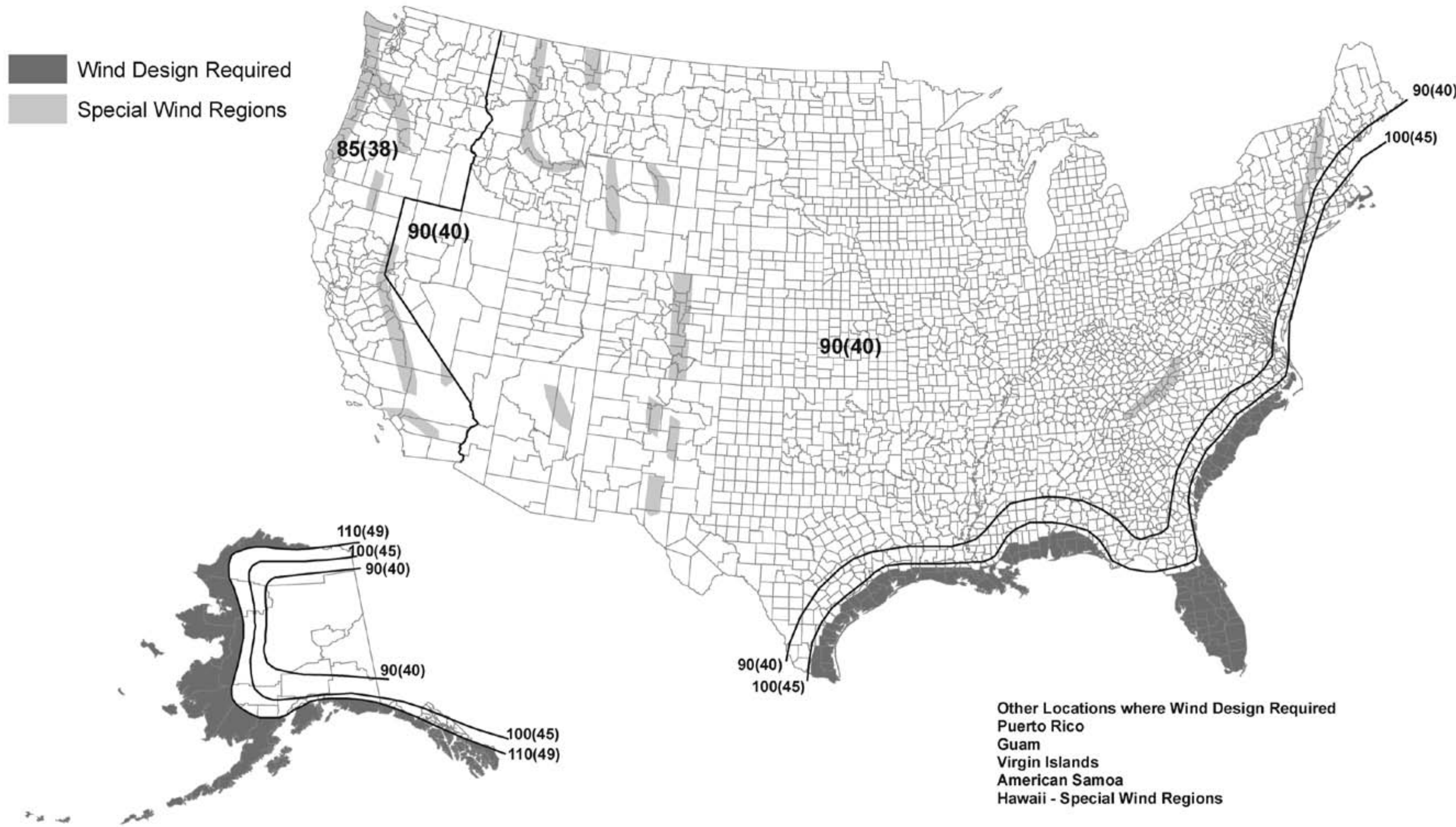


Figure 4-19B. Regions where wind design is required

SOURCE: IRC, USED WITH PERMISSION

These reports present detailed engineering discussions of building failure modes along with successful building performance guidance supplemented with design sketches. Please refer to these documents for specific engineering recommendations.

4.1.7 Seismic Forces

Seismic forces on a home and the structural elements of a foundation can be significant. Seismic forces may also trigger additional hazards such as landslides and soil liquefaction, which can increase the damage potential on a home. These forces act on structural components such as walls, roofs, connections, and foundations. Similar to wind forces, seismic forces should be considered in the design process at the same time as hydrostatic, hydrodynamic, impact, and building dead and live loads, and loads from other natural hazards such as hurricanes. Requirements for seismic design are normally available in locally adopted building codes. Requirements in ASCE 7 and model building codes such as the IBC are often the basis of seismic requirements contained in locally adopted building codes.

Figures 4-20 and 4-21 illustrate steps of a seismic design process that includes estimating seismic loads and determining the ability of existing structural components to withstand these loads.

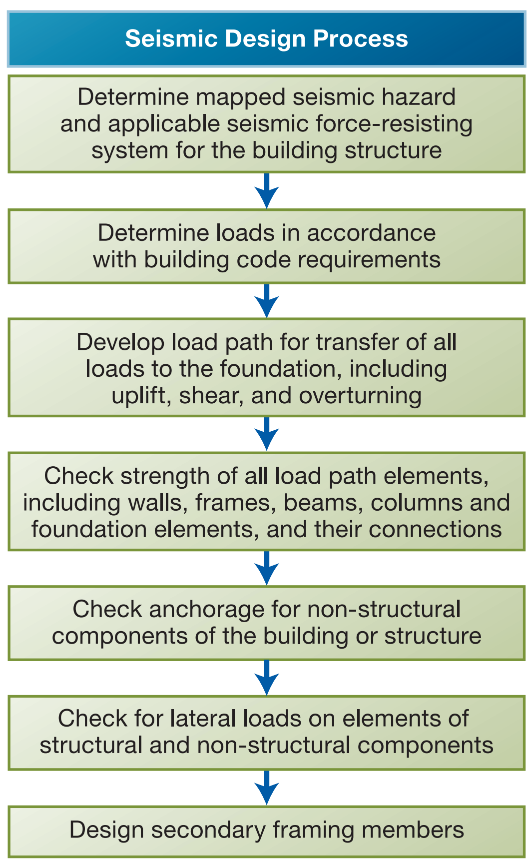


Figure 4-20. Seismic design process

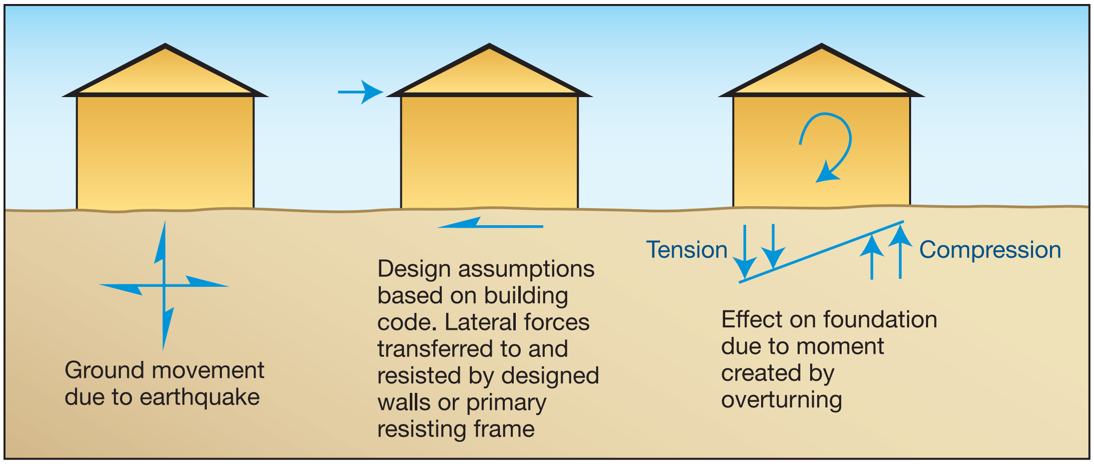


Figure 4-21. Seismic design causes and effects

When making repairs to a flood-damaged home or considering retrofitting structures to minimize the impact of future flooding events, there are certain practical steps that can be taken at the same time to reduce the chance of damage from other hazards. Earthquake protection steps can be divided into two categories: steps that deal with the building structure itself, and steps that can be taken with other non-structural parts of the building and its contents.

CROSS REFERENCE

If provisions of the local code do not address seismic loads or if a local code is not adopted for use, the designer should refer to the ASCE 7, *Minimum Design Loads for Buildings and Other Structures* or requirements of the International Building Code.

4.1.8 Combining Forces

Flood-related and non-flood-related forces need to be evaluated using applicable load combinations. Analysis of load combinations is covered in detail in Chapter 5 and ASCE 7.

CROSS REFERENCE

Refer to Section 5.2 for a detailed discussion of load combination scenarios and design methods.

4.1.9 Protection of the Structure

For protection of the building structure, the most important step is making sure the home is properly designed and constructed for seismic events. This includes proper design of the foundation and anchoring to the foundation. An engineered design will generally be required when the foundation of the house is raised above the BFE and the foundation is being considered to ensure the entire structure can withstand seismic forces.

CROSS REFERENCE

Additional information concerning the determination of flood-related forces is available in the flood design load criteria incorporated in Section 5 of ASCE 7, *Minimum Design Loads for Buildings and Other Structures*, and ASCE 24, *Flood Resistant Design and Construction*.

Key portions of masonry block foundations usually require strengthening by installing reinforcing bars in the blocks and then filling them with concrete grout. FEMA has developed a sample plan for strengthening a masonry block foundation wall. This type of work can be complicated and normally requires the expertise of a design professional such as an engineer or architect.

FEMA's *Technical Information on Elevating Substantially Damaged Residential Buildings in the Midwest* (1993d) provides procedures for determining seismic forces and recommendations for seismic retrofitting of a wood-frame structure. For more information on protecting a structure from seismic hazards, contact the appropriate FEMA Regional Office's Mitigation Division.

CROSS REFERENCE

The additional cost for seismic strengthening was estimated by FEMA (during the Midwest Floods of 1993) to range from 17-23 percent of the base repair cost for elevating a 1,000-square foot wood-frame structure on masonry foundation walls. FEMA prepared a methodology to estimate the costs of seismic retrofit projects described in FEMA 156, *Typical Costs for Seismic Rehabilitation of Existing Buildings* (FEMA, 1994).

4.1.10 Protection of Non-Structural Building Components and Building Contents

For non-structural building components and contents, earthquake protection usually involves simpler activities that homeowners can undertake themselves. These include anchoring and bracing of fixtures, appliances (e.g., hot water heaters and furnaces), chimneys, tanks, cabinets, shelves, and other items that may tip over or become damaged when subjected to earthquake ground shaking.

4.1.11 Land Subsidence

Subsidence of the land surface affects flooding and flood damages. It occurs in more than 17,000 square miles in 45 States and an area roughly the size of New Hampshire and Vermont combined. In 1991, the National Research Council estimated that annual costs in the United States from flooding and structural damage caused by land subsidence exceeded $125 million. Because the causes of subsidence vary, selected mitigation techniques are required in different situations.

Subsidence may result in sudden, catastrophic collapses of the land surface or in a slow lowering of the land surface. In either case, it can cause increased hazards to structures and infrastructure. In some cases, the causes of subsidence can be controlled.

CROSS REFERENCE

More information on land subsidence hazards can be obtained from the *Special Hazards Supplement to the CRS Coordinator's Manual*, dated 2006. This document is available through Flood Publications, *NFIP/CRS*, P.O. Box 501016, Indianapolis, Indiana 46250-1016. Telephone 317-845-2898.

Subsidence is typically a function of withdrawal of fluids or gases, the existence of organic soils, or other geotechnical factors; it requires an extensive engineering/geotechnical analysis. While NFIP regulations do not specifically address land subsidence, communities that develop mapping and regulatory standards addressing these hazards may receive flood insurance premium credits through the NFIP CRS. The designer should determine if a local community has mapped or enacted an ordinance covering this special hazard.

4.2 Geotechnical Considerations

Soil properties during conditions of flooding are important factors in the design of any surface intended to resist flood loads. These properties include:

- saturated soil forces (see Section 4.1.1.5);
- allowable bearing capacity;
- potential for scour;
- frost zone location;
- permeability; and
- shrink-swell potential.

CROSS REFERENCE`

Specific information on landslides and other geotechnical-related natural hazards can be found at http://landslides.usgs.gov/.

The computation of lateral soil forces and determination of soil bearing capacity are critical in the design of foundations. These forces plus the frost zone location and potential scour play an important role in determining the type of foundation to use. Likewise, the permeability and compactibility of soils are key factors in selecting borrow materials for backfill or levee construction.

Site investigations for soils include surface and subsurface investigations. Surface investigations can identify evidence of landslides, areas affected by erosion or scour, and accessibility for equipment needed for subsurface testing and construction. Surface investigations can also help identify the suitability or unsuitability of particular foundation styles based on the past performance of existing structures. Subsurface exploration provides invaluable data on soils at and below grade. The data are both qualitative (e.g., soil classification) and quantitative (e.g., bearing capacity). Although some aspects of subsurface exploration are discussed here, subsurface exploration is too complicated and site-dependent to be covered fully in one document. Consulting with geotechnical engineers familiar with the site is strongly recommended.

If unsure of local soil conditions, obtain a copy of the U.S. Department of Agriculture, NRCS *Soil Survey* of the general area. This survey provides valuable information needed to conduct a preliminary evaluation of the soil properties, including:

- type, location, and description of soil types;
- use and management of the soil types; and
- engineering and physical properties, including plasticity indexes, permeability, shrink/swell potential, erosion factors, potential for frost action, and other information.

This information can be compiled using Figure 4-22 to enable the designer to determine the suitability of the specific soil type to support the various retrofitting methods. It is important to note that, while the soil properties may not be optimum for specific retrofitting methods, facilities can often be designed to overcome soil deficiencies.

NOTE

The physical properties of soil are critical to the design, suitability, and overall stability of floodproofing measures. Therefore, the designer should consult a geotechnical engineer if the soil properties at a site do not support the use of the chosen retrofitting method. A geotechnical engineer should also be consulted for any information that cannot be obtained from the *Soil Survey* or the local office of the NRCS.

Geotechnical Considerations Decision Matrix

Owner Name:______________________ Prepared By:______________________

Address: ______________________ Date: ______________

Property Location: ______________________________________

Considerations	Elevation on Foundation Walls	Elevation on Fill	Elevation on Piers	Elevation on Posts and Columns	Elevation on Piles	Relocation	Dry Flood-proofing	Wet Flood-proofing	Floodwalls and Levees
Lateral Soil Pressure									
High									
Moderate									
Low									
Bearing Capacity									
High									
Moderate									
Low									
Potential for Scour									
High									
Moderate									
Low									
Shrink/ Swell Potential									
High									
Moderate									
Low									
Potential Frost Action									
High									
Moderate									
Low									
Permeability									
High									
Moderate									
Low									

Instructions: This matrix is designed to help the designer identify situations where soil conditions are unsuitable when applied to certain retrofitting measures, therefore eliminating infeasible measures. It is not intended to select the most suitable alternative. Instructions for use of this matrix follow:

1. Circle the appropriate description for each of the soil properties.
2. Use the NRCS *Soil Survey*, information from this and other reference books, and engineering judgment to determine which methods are Suitable (S) / Not Suitable (NS) for each soil property. Enter S or NS in each box.
3. Review the completed matrix and eliminate any retrofitting measures that are clearly unsuitable for the existing soil conditions.

Figure 4-22. Geotechnical Considerations Decision Matrix

The following sections begin a discussion of the various soil properties, providing the information necessary to fill out the Geotechnical Considerations Decision Matrix (Figure 4-22) and to understand the relationship between these soil properties and retrofitting measures.

4.2.1 Allowable Bearing Capacity

The weight of the structure, along with the weight of backfilled soil (if present), creates a vertical pressure under the footing that must be resisted by the soil. The term "allowable bearing pressure" refers to the maximum unit load that can be placed on a soil deposit without causing excessive deformation, shear failure, or consolidation of the underlying soil.

Bearing capacity has a direct effect on the design of shallow foundations. Soils with lower bearing capacities require proportionately larger foundations to effectively distribute gravity loads to the supporting soils. For deep foundations, like piles, bearing capacity has less effect on the ability of the foundation to support gravity loads because most of the resistance to gravity loads is developed by shear forces along the pile.

Bearing capacity is generally measured in pounds per square foot (lb/ft^2) and occasionally in tons per square foot. Soil bearing capacity typically ranges from 1,000 lb/ft^2 (relatively weak soils) to more than 10,000 lb/ft^2 (bedrock). The allowable bearing capacity is the ultimate bearing capacity divided by an appropriate factor of safety. The factor of safety depends on whether the soils have been tested. Soil-bearing-capacity testing will result in detailed soil characteristics producing a reasonable and accurate factor of safety. An appropriate factor of safety between 2 and 3 should be used if soil testing has not been completed. See Equation 4-16.

CROSS REFERENCE

An approach developed by FEMA during the elevation of substantially damaged homes in Florida and the Midwest is to reuse the existing footings, if allowed by code. Refer to FEMA 347, *Above the Flood: Elevating Your Floodprone House* (FEMA, 2000a) for details on elevation of structures.

Table 4-8 presents estimated allowable bearing capacities for various soil types to be used for preliminary sizing of footings only. The actual allowable soil bearing capacity should be determined by a soils engineer. Most local building codes specify an allowable bearing capacity to be utilized in design if the soil properties have not been specifically determined.

EQUATION 4-16: ALLOWABLE BEARING CAPACITY

$$Q_{BC} = \frac{Q_u}{FS} \qquad \text{(Eq. 4-16)}$$

where:

Q_{BC} = allowable bearing capacity (lb/ft^2)

Q_u = ultimate bearing capacity (lb/ft^2)

FS = factor of safety (as prescribed by code)

Table 4-8. Typical Allowable Bearing Capacity by Soil Type Shown in Table 4-4

Soil Type (Symbol)	Allowable Bearing Capacity (lb/ft^2)
Clay, Soft (CL, CH)	600 to 1,200
Clay, Firm (CL, CH)	1,500 to 2,500
Clay, Stiff (CL, CH)	3,000 to 4,500
Loose Sand, Wet (SP, SW, SM)	800 to 1,600
Firm Sand, Wet (SP, SW, SM, SC)	1,600 to 3,500
Gravel (GW, GP, GM, GC)	2,700 to 3,000

Once the allowable bearing capacity is determined by the soils engineer or a conservative estimate prescribed by code is made, the designer can determine the capacity of the existing foundation to support the expected loads. Depending on the outcome of that evaluation, the designer may need to supplement the existing footing to support the expected loading condition (i.e., keep the actual bearing pressure below the allowable bearing pressure of the soil) as a result of the retrofitting project.

The ability of soils to bear loads, usually expressed as shearing resistance, is a function of many complex factors, including some that are site-specific. A very significant factor affecting shearing resistance is the presence and movement of water within the soil. Under conditions of submergence, some shearing resistance may decrease due to the buoyancy effect of the interstitial water or, in the case of cohesive soils, to physical or chemical changes brought about in clay minerals.

NOTE

Certain types of soil – loose sands and soft clays (SP, SW, SM, SC, CL, CH) exhibit very poor bearing capacities when saturated; therefore, foundation, floodwall, and levee applications in those conditions would not be feasible without special treatment.

While there are many possible site-specific effects of saturation on soil types, some classes of soil can be identified that have generally low shearing resistances under most conditions of saturation. These include:

- fine, silty sands of low density that in some localities may suddenly compact when loaded or shaken, resulting in a phenomenon known as liquefaction;
- sand or fine gravel, in which the hydraulic pressure of upward-moving water within the soil equals the weight of the soil, causing the soil to lose its shear strength and become "quicksand," which will not support loads at the surface; and
- soils below the water table that have lower bearing capacity than the same soils above the water table.

WARNING

Attempts to construct water- or saturated soil-retaining/resisting structures without a thorough understanding of soil mechanics and analysis of on-site soils can result in expensive mistakes and project failure.

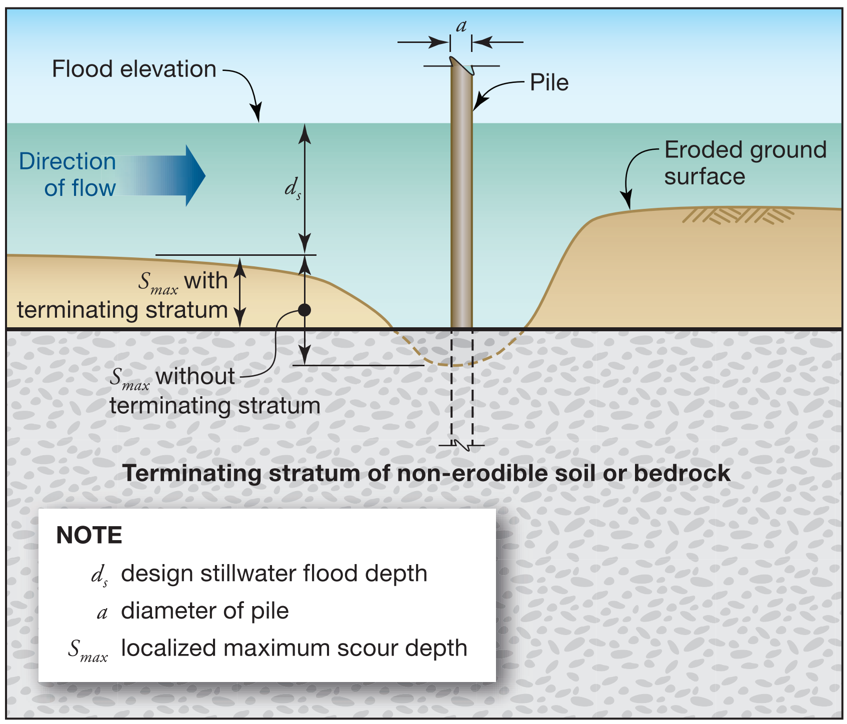
Other types of saturated soils may also have low shearing resistances under loads, depending on numerous site-specific factors such as slope, hydraulic head, gradient stratigraphic relationships, internal structures, and density. Generally, the soils noted above should not be considered suitable for structural support or backfill for retrofitting and, when they are known to be present, a soils engineer should be consulted for site-specific solutions.

4.2.2 Scour Potential

Erosion of fill embankments, levees, or berms depends on the velocity, flow direction, and duration of exposure. Scour is localized erosion caused by the entrainment of soil or sediment around flow obstructions, often resulting from flow acceleration and changing flow patterns due to flow constriction. Where flow impinging on a structure is affected by diversion and constriction due to nearby structures or other obstructions, flow conditions estimated for the calculation of depths of scour should be evaluated by a qualified engineer.

The effects of flood loads on buildings can be exacerbated by flood-induced erosion and localized scour and by long-term erosion, all of which can lower the ground surface around foundation elements and cause the loss of load-bearing capacity and loss of resistance to lateral and uplift loads. This can render structural retrofitting and resistive designs ineffective, possibly resulting in failure. Figures 4-23 and 4-24 illustrate scour at open foundation systems and ground level buildings.

Figure 4-23. Localized scour at piers, posts, and piles



Maximum potential scour is critical in designing an elevated foundation system to ensure that failure during and after flooding does not occur due to any loss in bearing capacity or anchoring resistance around the piers, posts, or piles. If a pier, post, or pile was not designed to withstand a maximum potential scour, and was exposed to scour from a flood event, the column will be subjected to loading in a condition it was not designed for, which may result in a failure of the foundation. If a pier, post, or pile were to have 4 feet of scour around its base, and the structural element was designed to have a depth of 5 feet, the point of fixity (depth into the ground where foundation is assumed fixed against rotation) would decrease significantly, and the flood depth at the column would increase significantly.

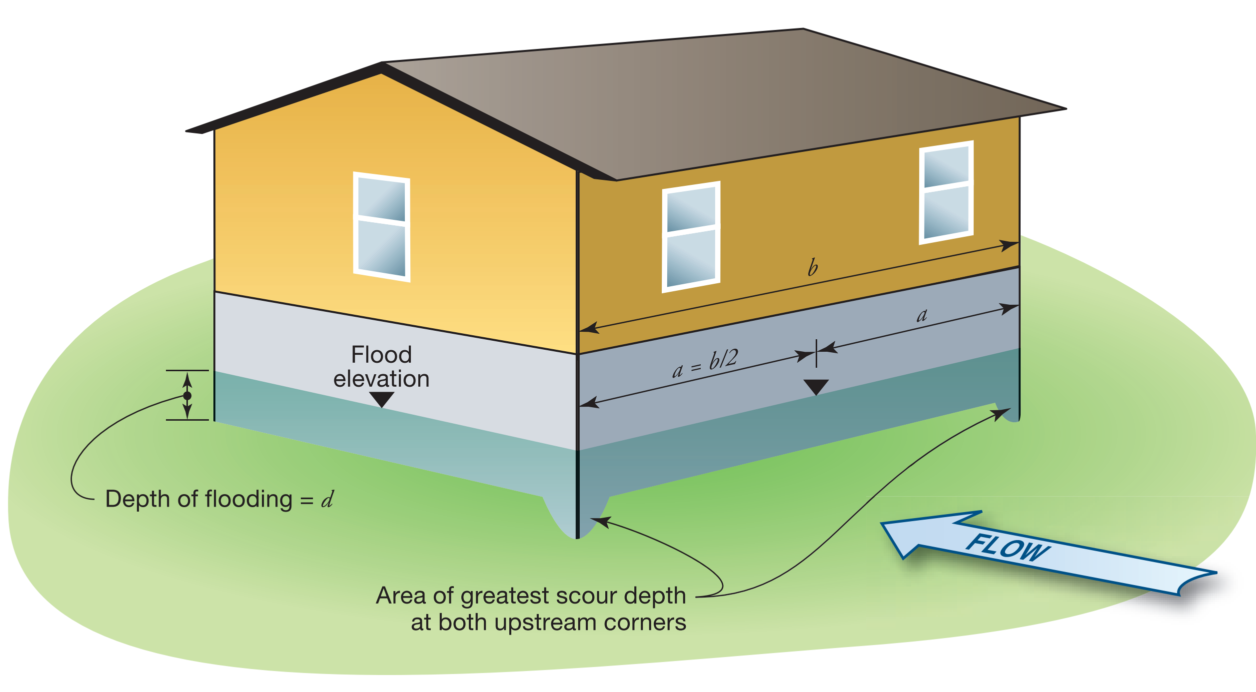


Figure 4-24. Scour action on a ground level building

The potential for foundation scour is a complex problem. Granular and other consolidated soils in which the individual particles are not cemented to one another are subject to scour, erosion, and transport by the force of moving water. The greater the velocity or turbulence of the moving water, the greater the scour potential. Soils that contain sufficient proportions of clay to be described as compact are more resistant to scour than the same grain sizes without clay as an intergranular bond. Likewise, soils with angular particle shapes tend to lock in place and resist scour forces.

NOTE

Resistance to scouring increases with clay content and/or the introduction of bonding agents, which help bond the internal particles of a soil together.

Shallow foundations in areas subject to flood velocity flow may be subject to scour and appropriate safeguards should be undertaken. These safeguards may include the use of different, more erosion-resistant soils, deeper foundations, surface armoring of the foundation and adjacent areas, and the use of piles or other foundations that present less of an obstruction to floodwater.

The calculation for estimating maximum potential scour depth at an elevated or ground-level foundation member (Equation 4-19) is based upon the foundation (or foundation member) shape and width, as well as the water velocity and depth, and type of soil.

Where elevation on fill is the primary retrofitting measure, embankments must be protected against erosion and scour. Scour at the embankment toe may be calculated as shown in Equation 4-17.

EQUATION 4-17: MAXIMUM POTENTIAL SCOUR AT EMBANKMENT TOE

$$S_{max} = d\left[1.1\left(\frac{a}{d}\right)^{0.4}\left(\frac{V}{(gd)^{0.5}}\right)^{0.33}\right] \qquad \text{(Eq. 4-17)}$$

where:

S_{max} = maximum potential depth of scour hole (ft)

d = depth of flow upstream of structure (ft)

a = diameter of pier, post, or pile or half the frontal length of the blockage (ft)

V = velocity of flow approaching the structure (ft/sec)

g = acceleration of gravity (equal to 32.2 ft/sec)

NOTE

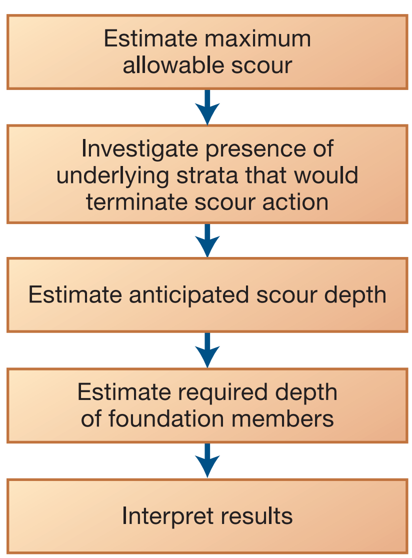
The factor "*a*" in Equation 4-17 is the diameter of an open foundation member or half of the width of the solid foundation perpendicular to flood flow.

The maximum potential scour depth predicted by the following equation represents a maximum depth that could be achieved if the soil material were of a nature that could be displaced by the water's action. However, in many cases, a stronger underlying stratum will terminate the scour at a more shallow elevation. Figure 4-25 illustrates the process of determining the potential scour depth affecting a foundation system.

WARNING

The scour information presented is the best available; however, there is not a general consensus within the scientific community that these scour equations are valid. Research continues into this area.

Figure 4-25. Process for estimating potential scour depth



Step 1: Estimate maximum allowable scour. The scour depth at square and circular pier, post, and pile foundation members can be calculated as shown in Equation 4-18.

EQUATION 4-18: LOCALIZED SCOUR AROUND VERTICAL PILE

$$S_{max} = 2.2a \qquad \text{(Eq. 4-18)}$$

where:

S_{max} = maximum potential depth of scour hole (ft)

a = diameter of a round foundation element, or the maximum diagonal cross section dimension for a rectangular element (ft)

NOTE

Equation 4-18 can also be used to approximate local scour beneath grade beams – set "a" equal to the depth (vertical thickness) of the grade beam.

Localized scour around vertical walls and enclosed areas (e.g., typical Zone A construction) can be greater than that around vertical piles and should be calculated as shown in Equation 4-19.

NOTE

Scour depths estimated with Equation 4-19 can be unrealistically high for coastal areas and should be capped at 10 feet of localized scour.

EQUATION 4-19: LOCALIZED SCOUR AROUND VERTICAL ENCLOSURE

$$S_{max} = d_s\left[2.2\left(\frac{a}{d_s}\right)^{0.65}\left(\frac{V}{(gd_s)^{0.5}}\right)^{0.43}\right]K \qquad \text{(Eq. 4-19)}$$

where:

S_{max} = maximum potential depth of scour hole (ft)

d_s = design stillwater flood depth upstream of the structure (ft)

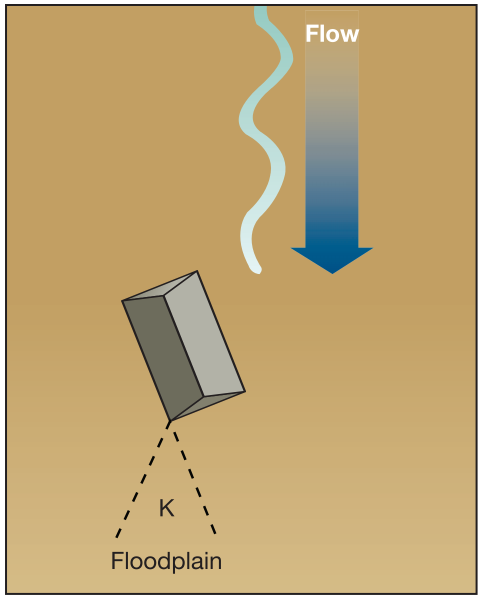
a = diameter of a round foundation element, or the maximum diagonal cross section dimension for a rectangular element (ft)

V = velocity of flow approaching the structure (ft/sec)

g = acceleration of gravity (equal to 32.2 ft/sec^2)

K = factor applied for flow angle of attack (see Figure 4-26)

Figure 4-26.
Flow Angle of Attack



The above scour equation applies to average soil conditions (2,000–3,000 lb/ft^2 bearing capacity). Average soil conditions would include gravels (GW, GP, GM, and GC), sands (SW, SP, SM, and SC), and silts and clays (ML, CL, MH, and CH). For loose sand and hard clay, the maximum scour values may be increased and decreased, respectively, to reflect their lower and higher bearing capacities. However, the assistance of a soils engineer should always be sought when making this adjustment, computing scour depths, and/or designing foundations subject to scour effects.

If a wall or foundation member is oriented at an angle to the direction of flow, a multiplying factor, *K*, can be applied to the scour depth to account for the resulting increase in scour as presented in Table 4-9.

Table 4-9. Scour Factor for Flow Angle of Attack, *K*

Angle of Attack	Length to Width Ratio of Structural Member in Flow: 4	8	12	16
0	1	1	1	1
15	1.15	2	2.5	3
30	2	2.5	3.5	4.5
45	2.5	3.5	4.5	5
60	2.5	3.5	4.5	6

Step 2: Investigate underlying soil strata. Once the maximum potential scour depth has been established, the designer should investigate the underlying soil strata at the site to determine if the underlying soil is of sufficient strength to terminate scour activities. Information from the NRCS *Soil Survey* may be used to make this assessment (http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm).

NOTE

The U.S. Department of Transportation recommends a factor of safety of 1.5 for predicting building scour depth.

Figure 4-27 illustrates a scour-terminating stratum. If an underlying terminating stratum does not exist at the site, the maximum potential scour estimate will become the anticipated scour depth. However, if an underlying terminating stratum exists, the maximum potential scour depth will be modified to reflect this condition, as shown in Step 3.

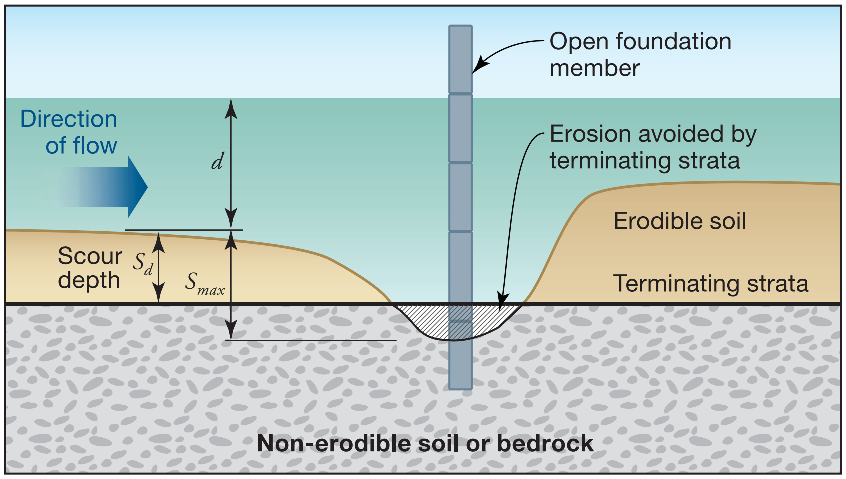


Figure 4-27. Terminating stratum

Step 3: Estimate anticipated scour depth. Based on the results of Step 2, the designer will determine the anticipated scour depth to be used in determining the depth to which the foundation element must be placed to resist scour effects. If a terminating stratum exists, the expected scour would stop at the depth at which this stratum starts, and the distance from this point to the surface is considered to be the potential scour depth, *(S_d)*. If no terminating stratum exists, the maximum potential scour *(S_{max})* computed earlier becomes the S_d.

Step 4: Estimate required depth of foundation members. Scour will increase the height above grade of the vertical member, since the grade level would be lowered due to erosion and scour (see Figure 4-28). As this occurs, the depth of burial *(D_b)* of the vertical foundation member also decreases an identical distance. This can result in a foundation failure because the loss of supporting soils would change the assumed conditions

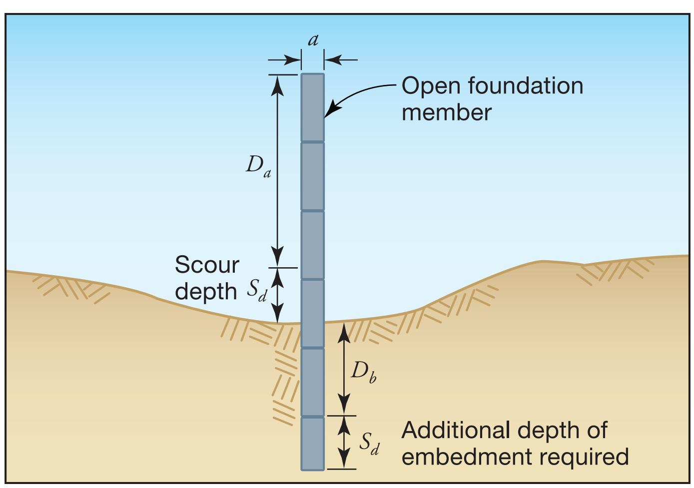


Figure 4-28. Additional embedment for foundation member

under which the elevated foundation system was designed. To account for this, the vertical foundation member depth used for the purpose of determining an acceptable design must be increased by the amount of S_d.

Step 5: Interpret results. Foundations, footings, and any supporting members should be protected at least to the anticipated scour depth. If the structural member cannot be buried deeper than the anticipated scour depth, the member should be protected from scour by placing rip-rap (or other erosion-resistant material) around the member, or by diverting flow around the foundation member with grading modification or construction of an independent barrier (floodwall or levee). For situations in which the anticipated scour depth is minimal, the designer should use engineering judgment to determine the required protective measures. Whenever the designer is unsure of the appropriate action, a qualified geotechnical engineer should be consulted.

CROSS REFERENCE

Local building codes generally specify the depth of the zone of maximum frost penetration. In the absence of guidance in the local building code, refer to the NWS or the NRCS *Soil Survey*.

4.2.2.1 Frost Zone Considerations

Because certain soils under specific conditions expand upon freezing, the retrofitting designer must consider the frost heave impact in the design of shallow foundations. When frost-susceptible soils are in contact with moisture and subjected to freezing temperatures, they can imbibe water and undergo very large expansions (both horizontally and vertically). Such heave or expansion exerts forces strong enough to move and/or crack adjacent structures (foundations, footings, etc.). The thawing of frozen soil usually proceeds from the top downward. The melted water cannot drain into the frozen subsoil, and thus becomes trapped, possibly weakening the soil. Normally, footing movements caused by frost action can be avoided by placing part of a foundation below the zone of maximum frost penetration.

4.2.2.2 Permeability

A principal concern for the construction of retrofitting measures such as floodwalls and levees are the properties of the proposed fill material and/or underlying soils. These properties will have an impact on stability and will determine the need for seepage and other drainage control measures.

Since most retrofitting projects are constructed using locally available materials, it is possible that homogenous and impermeable materials will not be available to construct embankments and/or backfill floodwalls and foundations. Therefore, it is essential that the designer determine the physical properties of the underlying and borrowed soils.

Where compacted soils are highly permeable (i.e., sandy soils), significant seepage through an embankment and under a floodwall foundation can occur. Various soil types and their permeabilities are provided in Table 4-10.

NOTE

While impervious cutoffs such as compacted impervious core, sheet pile metal curtains, or cementitious grout curtains can be designed to reduce or eliminate seepage, their costs are beyond the financial capabilities of most homeowners. However, several lower-cost measures to control seepage include pervious trenches, pressure relief wells, drainage blankets, and drainage toes.

Table 4-10. Typical Values of Coefficient of Permeability *K* for Soils

Soil Type and Description	Symbol	Typical Coefficient of Permeability (ft/day)
Well-graded clean gravels, gravel-sand mixtures	GW	75
Poorly graded clean gravels, gravel-sand-silt	GP	180
Silty gravels, poorly graded gravel-sand-silt	GM	1.5 x 10-3
Clayey gravels, poorly graded gravel-sand-clay	GC	1.5 x 10-4
Well-graded clean sands, gravelly sands	SW	4.0
Poorly graded clean sands, sand-gravel mix	SP	4.0
Silty sands, poorly graded sand-silt mix	SM	2.0 x 10-2
Sand-silt clay mix with slightly plastic fines	SM-SC	3.0 x 10-3
Clayey sands, poorly graded sand-clay mix	SC	7.5 x 10-4
Inorganic silts and clayey silts	ML	1.5 x 10-3
Mixture of inorganic silt and clay	ML-CL	3.0 x 10-4
Inorganic clays of low to medium plasticity	CL	1.5 x 10-4
Organic silt and silt-clays, low plasticity	OL	Quite Variable
Inorganic clayey silts, elastic silts	MH	1.5 x 10-4
Inorganic clays of high plasticity	CH	1.5 x 10-2
Organic clays and silty clays	OH	Quite Variable

1 cm/sec = 24,680 ft/day = 2 ft/min 1 ft/year = 1 x 10-6 cm/sec

The coefficient of permeability provides an estimate of ability of a specific soil to transmit seepage. It can be used (Equation 4-20) to make a rough approximation of the amount of foundation underseepage. Equation 4-20 may be used in lieu of Equation 4-14 for large levee/floodwall applications when the coefficient of permeability for the specific site soil is known.

EQUATION 4-20: VOLUME OF SEEPAGE

$$Q = k i_{hg} A \qquad \text{(Eq. 4-20)}$$

where:

Q = the discharge in a given unit of time (ft^3/unit of time)

k = coefficient of permeability for the soil foundation (ft/unit of time)

i_{hg} = hydraulic gradient *(h/L)* which is the difference in head between two points divided by the length of path between two points

A = gross area of the foundation through which flow takes place (ft^2)

WARNING

It is very important that the designer keep the units in this equation consistent. The results of Equation 4-20 depend on the homogeneity of the foundation and the accuracy of the coefficient of permeability. The results should be considered as an indication only of the order of magnitude of seepage through a foundation.

4.2.2.3 Shrink-Swell Potential

As mentioned earlier in this chapter, due to the continual shrink and swell of expansive soil backfills and the variation of their water content, the stability and elevation of these soils and overlaying soil layers may vary considerably. These characteristics make the use of these soils in engineering/construction applications imprudent. The NRCS *Soil Survey* for a specific area offers guidance on the shrink-swell potential of each soil group in the area as well as guidance on the suitability of their use in a variety of applications, including engineering, construction, and water retention activities. Table 4-10 provides typical values for the coefficient of permeability (K) for soils. If the designer is unsure of the type or nature of soil at the specific site, a qualified soils engineer should be contacted for assistance.

NOTE

Soils that exhibit severe shrinking-swell characteristics include clays and clay mixtures such as Soil Types CH, CL, ML-CL, SC, and MH.

The physical soil parameters at the retrofitting and potential borrow sites are an important design consideration. Homeowners and designers should clearly understand that the advice of a professional soils engineer is vital when planning retrofitting measures that are not ideal for the physical soil parameters at a given site.

Chapter 5 provides guidance on how to apply the anticipated loads and calculate load combinations developed in this chapter to the existing site/structure. Examples for calculating flood loads, other anticipated loads, and load combinations can be found in Appendix C.